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Chapter 11

DAMAGE TO STRUCTURES [redacted]

[redacted] INTRODUCTION [redacted]

[redacted] The two principal phenomena associated with nuclear explosions that result in damage to structures are blast and shock, and thermal radiation. In general, structures that are located above ground are more susceptible to damage by air blast than by ground shock; however, if the depth of the burst is sufficiently great, ground shock may be the predominant damage producing phenomenon. Buried structures, on the other hand, are more susceptible to damage from ground shock, either air-induced or direct-transmitted. Fires also are a primary cause of damage to structures. The fires may result from ignition of material by the thermal pulse from the explosion, or they may be initiated as a consequence of blast damage, e.g., by the rupture of gas lines.

[redacted] This chapter contains seven sections. The first section provides the methods for evaluation of the vulnerability of conventional surface or aboveground structures. Additional information concerning these analyses is provided in Appendix C. The free-field air blast environmental data that are required for use with the methodology described in Appendix C are contained in Section I of Chapter 2.

[redacted] Section II provides the methodology for evaluating the vulnerability of underground structures. Section III describes the shock vulnerability of equipment and personnel. Both Sections II and III of this chapter are closely related to Section III of Chapter 2.

[redacted] Section IV provides a description of the damage mechanisms for dams and harbor installations. Section V contains procedures for estimating vulnerability of petroleum, oil, and lubricant (POL) storage tanks, while Section VI contains similar estimates for field fortifications.

[redacted] The first six sections of this chapter are concerned primarily with blast and shock damage to structures. Section VII provides a discussion of fire in urban areas that could result from a nuclear explosion. Selected examples of analyses that have been performed are provided in Section VII. Section III of Chapter 9 contains additional information concerning thermal radiation damage to materials that might ultimately result in structural damage.

SECTION I

[redacted] DAMAGE TO ABOVEGROUND STRUCTURES [redacted]

[redacted] Damage to surface structures can be caused by the blast and thermal effects from a nuclear detonation. Air blast may cause damage that ranges from the breaking of windows and cracking of plaster to virtually complete destruction. The nature and extent of this damage will depend upon the weapon yield, the height of burst, the distance from ground zero to the structure, and the nature of the structure. The characteristics of the structure that affect the magnitude and type of damage are the strength, ductility, shape, mass of the structural frame and of the wall and roof coverings, and the number and size of openings.

[redacted] The magnitudes of the vertical and horizontal blast loadings on the surfaces of a structure vary with the angle of elevation from the structure to the burst. When a structure is directly underneath the burst point (at ground zero),

[redacted]

[redacted] the roofs and floors may be dished in or destroyed, and the walls may collapse, but there is little or no tendency to displace the structure laterally. Farther from ground zero, the horizontal component of the loading becomes important. Under these circumstances damage generally is caused by a lateral distortion of the frame. For most structures, the damage produced by a given overpressure increases with weapon yield because of the corresponding increase in the duration of the loading.

[redacted] Direct thermal radiation causes fires by ignition of kindling fuels. These ignitions may occur well beyond the limits of significant blast damage. Blast may also damage structures by secondary effects, such as by fires initiated by short circuits, ruptured gas mains and ruptured or overturned stoves and furnaces. Fire in urban areas is described separately in Section VII.

[redacted] The effects of ground shock, either directly transmitted through the ground or induced by the air blast wave, on the structural damage of aboveground structures is negligible in comparison to the damage produced by air blast overpressures.

[redacted] AIR BLAST EFFECTS [redacted]

11-1 Nature of Loading [redacted]

[redacted] The blast loads on a structure depend on the free-field blast phenomena and the geometry, orientation, and environment of the structure.

[redacted] A large essentially closed structure with walls that remain intact throughout most of the load duration is primarily sensitive to the diffraction phase (see Section II, Chapter 9), since most of the translational load is applied during this period. When a blast wave strikes this type of structure, it is reflected, thereby creating pressures greater than the incident overpressure. Subsequently, the reflected pressure decays to that of the free-field air blast wave. As the blast wave progresses, it engulfs the structure and soon exerts overpressures on all sides. Before the blast wave reaches the rear face, pressures on the front exert translational forces in the direction of blast wave propagation. After the wave reaches the rear face, the pressures on the rear tend to counter the pressures on the front. For smaller structures, the blast wave reaches the rear face more quickly, and the pressure differential exists for a shorter time. Thus, the net translational loading during the diffraction phase depends heavily on structural dimensions. A longer duration blast wave does not materially change the magnitude of the net translational loading during the diffraction phase or the damage resulting therefrom. In other words, the structure is primarily sensitive to the peak blast wave overpressures.

[redacted] During the diffraction phase and until the entire blast wave has passed, dynamic pressures are also exerted on structures. Dynamic pressure loading is commonly referred to as drag loading. In the case of a large closed structure, the drag phase loading is small relative to the overpressure loading during the diffraction phase. The drag phase assumes greater relative importance for smaller structures. For small area components, such as the frame of a structure after removal of siding, the translational load applied as a result of the drag phase is much greater than the net translational loading from overpressures during the diffraction phase. The drag phase is the dominant factor in producing damage to frame buildings, whose siding is removed during the diffraction phase. The net load on bridges during the diffraction phase is applied for an extremely short time, while the drag phase continues until the entire blast wave has passed the structure. Since the drag phase duration is closely related to the duration of the blast wave rather than to the overall dimensions of the structure, damage depends not only on peak dynamic pressure but also on the duration of the positive phase of the dynamic pressure.

[redacted]

Thus, damage to this type of structure depends on weapon yield as well as on peak overpressure.

[redacted] In constructing the isodamage curves that will be described later (paragraph 11-4), the shapes of both the free-field overpressure and dynamic pressure pulses were assumed to be ideal (see Section I, Chapter 2), and simplified representations of these were used. Because of the lack of available quantitative data, nonideal wave forms, nonzero rise times, and precursor formations were not considered in the calculations. The idealized functions that were assumed to act on the structures are shown schematically in Figures 11-1a through 11-1d. The method by which these functions were generated is described in Appendix C. The structure numbers shown in Figure 11-1 refer to the structures that are described and numbered in Tables 11-1 and 11-2 (see paragraph 11-2).

11-2 Nature of Damage [redacted]

[redacted] Damage may range from none to light, moderate, or severe. Descriptions of the several degrees of damage for the various types of structures are given in Tables 11-1 and 11-2.* Table 11-1 lists those types of structures that generally are affected primarily by air blast overpressure during the diffraction phase, while Table 11-2 lists those types of structures that are primarily sensitive to drag loading. A major factor to consider in assessing structural damage is the effect of the damage on continued operations within the structure. If rugged equipment is mounted on a foundation at ground level, major distortion or even collapse of a structure may not preclude operation of the equipment. Conversely, if the equipment is tied in with the structural frame, distortion of the structure may prevent or seriously affect operability of the equipment. No satisfactory general method has been developed for relating damage of structures to the operational equipment contained within them. This relationship may be established for particular cases of interest on an individual basis. In general, severe structural damage, approaching collapse, will entail a major reduction in operating capability. Damage to structures has been divided into three major categories as follows:

- *Severe damage.* That degree of structural damage that precludes further use of a structure for the purpose for which it is intended without essentially complete reconstruction. Generally, collapse of the structure is implied.
- *Moderate damage.* That degree of structural damage to principal load-carrying members (trusses, columns, beams, and load-carrying walls) that precludes effective use of a structure for the purpose for which it is intended until major repairs are made.
- *Light damage.* That degree of damage that results in broken windows, slight damage to roofing and siding, blowing down of light interior partitions, slight cracking of curtain walls in buildings, and as described in Tables 11-1 and 11-2 for other structures.

11-3 Structural Characteristics [redacted]

[redacted] The categorization of structures as being either diffraction sensitive or drag sensitive is justified in comparatively few cases. Most real surface structures are affected by, and respond to, loading during both phases. The structural characteristics significant in the determination of loading, response, and damage are: the strength, ductility, shape, and mass of the structural frame and of the wall and roof coverings, as well as the number and size of the openings.

[redacted] At one extreme is a structure with solid walls and roof that have resistances equal to or

*[redacted] The structure numbers in Tables 11-1, 11-2, and some succeeding tables in this section correspond to the figure number that portrays the isodamage curves for that structure (paragraph 11-4).

[redacted]

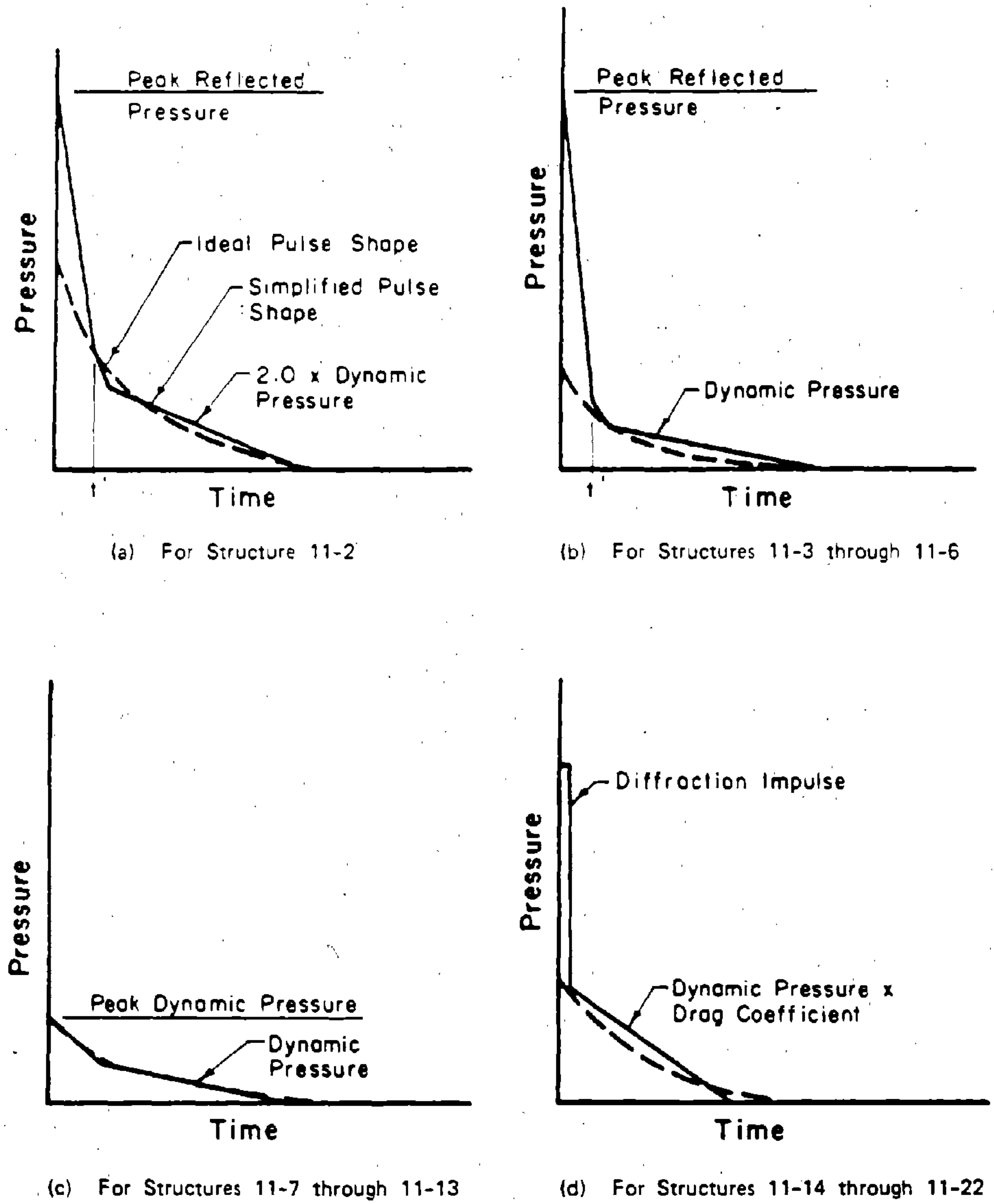

Figure 11-1. [redacted] Idealized Loading Functions [redacted]

[redacted]

Table 11-1. [redacted] Damage to Types of Structures Primarily Affected by Blast Wave Overpressure During the Diffraction Phase [redacted]

Structure and Figure Number	Description of Structure	Description of Damage: Severe	Moderate	Light
11-2	Multistory reinforced concrete building with reinforced concrete walls blast resistant designed for 30-psi Mach region pressure from 1 Mt, no windows	Walls shattered, severe frame distortion, incipient collapse	Walls breached or on the point of being so, frame distorted, entranceways damaged, doors blown in or jammed, extensive spalling of concrete.	Some cracking of concrete walls and frame
11-3	Multistory reinforced concrete building with concrete walls, small window area, three to eight stories.	Walls shattered, severe frame distortion, incipient collapse	Exterior walls severely cracked. Interior partitions severely cracked or blown down. Structural frame permanently distorted, extensive spalling of concrete.	Windows and doors blown in, interior partitions cracked
11-4	Multistory wall-bearing building, brick apartment house type, up to three stories.	Collapse of bearing walls, resulting in total collapse of structure	Exterior walls severely cracked interior partitions severely cracked or blown down.	Windows and doors blown in, interior partitions cracked.
11-5	Multistory wall-bearing building, monumental type, up to four story.	Collapse of bearing walls, resulting in collapse of structure supported by these walls. Some bearing walls may be intervening walls so that part of the structure may receive only moderate damage.	Exterior walls facing blast severely cracked, interior partitions severely cracked with damage toward far end of building possibly less intense.	Windows and doors blown in, interior partitions cracked
11-6	Wood frame building, house type, one or two stories.	Frame shattered resulting in almost complete collapse.	Wall framing cracked Roof severely damaged, interior partitions blown down.	Windows and doors blown in, interior partitions cracked.

[redacted]

Table 11-2. [redacted] Damage to Types of Structures Primarily Affected by Dynamic Pressure During the Drag Phase [redacted]

Structure and Figure Number	Description of Structure	Description of Damage		
		Severe	Moderate	Light
11-7	Light steel-frame industrial building, single story, with up to 5-ton crane capacity; low strength walls which fail quickly	Severe distortion or collapse of frame	Minor to major distortion of frame, cranes, if any, not operable until repairs made.	Windows and doors blown in, light siding ripped off
11-8	Heavy steel-frame industrial building, single story, with 25 to 50-ton crane capacity lightweight, low strength walls which fail quickly.	Severe distortion or collapse of frame	Some distortion to frame, cranes not operable until repairs made.	Windows and doors blown in, light siding ripped off.
11-9	Heavy steel-frame industrial building, single story, with 60 to 100-ton crane capacity, lightweight low strength walls which fail quickly.	Severe distortion or collapse of frame.	Some distortion to frame, cranes not operable until repairs made.	Windows and doors blown in, light siding ripped off.
11-10	Multistory steel-frame office-type building, 3 to 10 stories. Lightweight low strength walls which fail quickly, earthquake resistant construction.	Severe frame distortion, incipient collapse.	Frame distorted moderately, interior partitions blown down.	Windows and doors blown in, light siding ripped off, interior partitions cracked.
11-11	Multistory steel-frame office-type building. 3 to 10 stories. Lightweight low strength walls which fail quickly, nonearthquake resistant construction.	Severe frame distortion, incipient collapse	Frame distorted moderately, interior partitions blown down.	Windows and doors blown in, light siding ripped off, interior partitions cracked.
11-12	Multistory reinforced concrete frame office-type building, 3 to 10 stories; lightweight low strength walls which fail quickly, earthquake resistant construction.	Severe frame distortion, incipient collapse.	Frame distorted moderately, interior partitions blown down, some spalling of concrete.	Windows and doors blown in, light siding ripped off, interior partitions cracked.

[redacted]

Table 11-2. [redacted] (Continued)

Structure and Figure Number	Description of Structure	Description of Damage: Severe	Moderate	Light
11-13	Multistory reinforced concrete frame office-type building, 3 to 10 stories, lightweight low strength walls which fail quickly, nonearthquake resistant construction	Severe frame distortion, incipient collapse.	Frame distorted moderately, interior partitions blown down, some spalling of concrete.	Windows and doors blown in, light siding ripped off, interior partitions cracked
11-14	Highway truss bridges, 4-lane, spans 200 to 400 ft, railroad truss bridges, double track, ballast floor, spans 200 to 400 ft	Total failure of lateral bracing or anchorage collapse of bridge	Substantial distortion of lateral bracing or slippage on supports, significant reduction in capacity of bridge	Capacity of bridge not significantly reduced, slight distortion of some bridge components
11-15	Highway truss bridges, 2-lane, spans 200 to 400 ft, railroad truss bridges, single track ballast or double track open floors, spans 200 to 400 ft; railroad truss bridges, single track open floor, span 400 ft	(Ditto)	(Ditto)	(Ditto)
11-16	Railroad truss bridges, single track open floor, span 200 ft.	(Ditto)	(Ditto)	(Ditto)
11-17	Highway girder bridges, 4-lane through, span 75 ft.	(Ditto)	(Ditto)	(Ditto)
11-18	Highway girder bridges, 2-lane deck, 2-lane through, 4-lane deck, span 75 ft, railroad girder bridges, double-track deck, open or ballast floor, span 75 ft; railroad girder bridges, single or double track through, ballast floors, span 75 ft.	Total failure of lateral bracing or anchorage, collapse of bridge.	Substantial distortion of lateral bracing or slippage on supports, significant reduction in capacity of bridge.	Capacity of bridge not significantly reduced, slight distortion of some bridge components.

Table 11-2. [redacted] (Concluded)

Structure and Figure Number	Description of Structure	Description of Damage: Severe	Moderate	Light
11-19	Railroad girder bridges, single track deck, open or ballast floors, span 75 ft, railroad girder bridges, single or double track, through, open floors, span 75 ft.	Total failure of lateral bracing or anchorage, collapse of bridge	Substantial distortion of lateral bracing or slippage on supports, significant reduction in capacity of bridge.	Capacity of bridge not significantly reduced, slight distortion of some bridge components.
11-20	Highway girder bridges, 2-lane through, 4-lane deck or through, span 200 ft, railroad girder bridges, double track deck or through, ballast floor, span 200 ft.	(Ditto)	(Ditto)	(Ditto)
11-21	Highway girder bridges, 2-lane deck, span 200 ft, railroad girder bridges, single track deck or through, ballast floors, span 200 ft, railroad girder bridges, double track deck or through, open floors, span 200 ft.	(Ditto)	(Ditto)	(Ditto)
11-22	Railroad girder bridges single track deck or through, open floors, span 200 ft.	(Ditto)	(Ditto)	(Ditto)
11-23	Floating bridges, U.S. Army standard M-2 and M-4, random orientation.	[redacted]	[redacted]	[redacted]

DNA (2)(1)

[redacted]

greater than that of the structural frame. Such a structure is primarily diffraction sensitive. At the other extreme is a structure covered with light frangible material (or virtually no walls) which is blown off under the initial blast impact leaving a skeleton frame sensitive only to drag forces. The majority of structures, whose walls have geometrical and strength characteristics sufficient to impart a certain amount of force to the structural frame before failing during the diffraction phase, lie between these two.

[redacted] As mentioned previously, the response of a structure, and consequently the nature and extent of damage, depends on the strength, ductility, and mass of the structure or structural element. Mass is significant in that it affects primarily the natural period of vibration. In general, as the mass increases, the period of vibration increases, and the maximum pressure of a given duration required to produce a specified amount of damage also increases, although these variations are not linear.

[redacted] The yield strength of a structure is of obvious significance, but of almost equal importance is ductility. Ductility is a measure of the ability of a structure, or element thereof, to deform beyond the limit of elasticity without fracturing. Ductility, therefore, is related directly to energy absorbing capacity and has a strong effect on the blast resistance afforded by a given structural yield strength. For a given yield strength, as the ductility increases, the maximum pressure of a given duration necessary to produce failure also increases, though not linearly.

[redacted] For each building type listed in Tables 11-1 and 11-2, the structural characteristics are sufficiently similar that structures of a given type are considered to respond in approximately the same way under identical loading conditions, despite a recognized variability of unknown amount for each type. The structural parameters taken as being characteristic, or typical, of each building type are shown in Table 11-3.

[redacted] Similar characteristics cannot be given conveniently for bridges because the significant mass, resistance, and dynamic parameters vary so greatly with span, even within each type of bridge.*

11-4 Isodamage Curves [redacted]

[redacted] Isodamage curves, which are functions of weapon yield, distance, and height-of-burst, are presented in Figures 11-2 through 11-23, for various types of structures for weapons ranging in yield from 0.01 kt to 30 Mt. It should be noted, however, that for structures where isodamage curves are given for low-yield weapons – ranging from 0.01 kt to about 3 kt – severe damage as read from the curve may imply severe damage to only a portion of the structure. For any yield, a structure which has a dimension greater than about half the damage distance given in the isodamage curves probably will not suffer total collapse.

[redacted] The character of damage will vary with the type of structure and with the position of the weapon with respect to the target structure. For an overhead burst, building roof systems may be dished in, walls cracked, partitions displaced, etc. to produce moderate damage; whereas for severe damage, the floor systems in general will collapse. Little or no lateral distortion of the frame may be expected from an overhead burst. For configurations other than overhead or near-overhead bursts, the damage to buildings will be largely due to lateral distortion of the frame and walls, rather than distortion of the floor system as discussed above.

[redacted] When subjected primarily to horizontally applied blast pressures, damage to bridges generally will result from failure of the lateral bracing systems or from failure of the anchorage connections and subsequent transverse sliding on

*[redacted] A more detailed discussion of bridge characteristics may be found in *Methods for Determining the Vulnerability of Selected Types of Bridges* (See Bibliography).

[redacted]

Table 11-3. [redacted] Building Parameter Values [redacted]

Structure and Figure Number	Range of Structural Parameters: Period of Vibration, T (sec)	Ductility Factor, μ*	Static Yield Resistance, q_y (psi)†	Clearing Distance S (feet)‡
11- 2	0.10-0.15	2- 5	55-80	30
11- 3	0.24-0.36	3- 7	2.5-3.5	5
11- 4	0.12-0.18	2- 5	1.0-1.6	3
11- 5	0.12-0.18	2- 5	3.0-5.0	5
11- 6	0.20-0.30	2- 6	0.4-0.6	3
11- 7	0.30-1.20	5-15	0.5-1.0	0
11- 8	0.20-0.80	5-10	0.7-2.5	0
11- 9	0.20-0.40	5-10	1.5-5.0	0
11-10	0.40-0.80	5-15	3.0-6.0	0
11-11	0.40-0.80	5-15	1.0-3.0	0
11-12	0.25-0.75	5-10	3.5-6.0	0
11-13	0.25-0.75	5-10	1.5-3.5	0

*Ratio of deflection corresponding to severe damage to deflection at yielding.

†Static pressure on exposed area causing yielding.

‡Shortest distance from stagnation point to a clear edge-variation not considered.

[redacted]

their supports. Under some circumstances, high, very narrow truss bridges may fail by overturning at pressures slightly less than those associated with failure through the two more common mechanisms. However, in view of the uncertainties involved, overturning is not considered as a possible failure mode in the isodamage curves for bridges.

[redacted] For an overhead burst, bridge response at a given pressure level is extremely sensitive to the height of the bridge above ground surface. Hence, since this additional parameter could not be represented on the isodamage curves, these curves were terminated at ranges at which the mode of damage was thought to change from transverse failure to vertical failure.*

[redacted] The degree of damage to a structure, caused by the air blast from a nuclear explosion depends on a great many parameters including those that affect the nature of the loading and those that affect the dynamic properties and resistance of the structure. Because all of these parameters are subject to variation for structures within a particular classification, detailed calculations for particular structures are quite com-

*[redacted] Failure of bridges under vertically applied loads may be investigated for individual cases by methods given in *Methods for Determining the Vulnerability of Selected Types of Bridges* (See Bibliography).

[redacted]

Table 11-4. [redacted] Reference List of Isodamage Curves for Various Types of Bridges [redacted]

Bridge Type	Span in Feet	Isodamage Curves (If two figures are shown, interpolate between them on the basis of span.)
Highway truss bridges (deck or through):		
4-lane	200–400	Fig. 11-14
2-lane	200–400	Fig. 11-15
Railroad truss bridges (deck or through):		
Double track, ballast floor	200–400	Fig. 11-14
Double track, open floor	200–400	Fig. 11-15
Single track, ballast floor	200–400	Fig. 11-15
Single track, open floor	200–400	Figs. 11-15 and 11-16
Highway girder bridges:		
4-lane through	75-200	Figs. 11-17 and 11-18
4-lane deck	75-200	Figs. 11-18 and 11-20
2-lane through	75-200	Figs. 11-18 and 11-20
2-lane deck	75-200	Figs. 11-18 and 11-21
Railroad girder bridges:		
Double track through, ballast floor	75-200	Figs. 11-18 and 11-20
Double track through, open floor	75-200	Figs. 11-19 and 11-21
Double track deck, ballast floor	75-200	Figs. 11-18 and 11-20
Double track deck, open floor	75-200	Figs. 11-18 and 11-21
Single track through, ballast floor	75-200	Figs. 11-18 and 11-21
Single track through, open floor	75-200	Figs. 11-19 and 11-22
Single track deck, ballast floor	75-200	Figs. 11-19 and 11-21
Single track deck, open floor	75-200	Figs. 11-19 and 11-22

[redacted]

plex. A number of studies have shown, however, that the major influences of these various parameters are taken into account adequately if simple conventionalized loading and resistance functions for the structures are considered.

[redacted] The isodamage curves presented in this section may be used to predict the conditions under which severe damage, as described in paragraph 11-2, may be expected. These predictions are made as functions of yield, height of burst, and horizontal distance to the target from ground zero.

[redacted] Since bridge vulnerability frequently is a function of span, it will often be necessary to interpolate between two sets of isodamage curves. For convenience, a reference list of isodamage curves for various types of bridges is given in Table 11-4.

[redacted]

[redacted] The significant structural parameters for each building type were represented by mean values of yield resistance, ductility factor, and period of vibration. The values of these parameters, which were given in Table 11-3, were based on available structural data and judgment from past experience.

[redacted] The reasonableness of the damage levels as portrayed by the isodamage curves contained herein has been verified by comparison to overpressures corresponding to severe damage for a number of actual buildings for which blast data are available from Hiroshima and Nagasaki, field tests, and more exact calculations. No comparable data are available with which to compare the bridge results.

[redacted] The isodamage curves in Figures 11-2 through 11-23 are drawn for 50 percent probability of attaining severe damage to the structure described in each figure. An estimate of the scaled distance at which there is a 50 percent probability of attaining moderate damage to a structure may be calculated by multiplying the distance obtained from the appropriate isodamage curve in Figures 11-2 through 11-23 by a factor for the appropriate structure and yield from Table 11-5. Where a factor of 1.0 is shown for a particular combination of yield and structure type, the distance at which moderate damage is expected to occur is expected to be less than 10 percent different than the distance at which severe damage is expected. Procedures for calculating damage distances for other probabilities of severe and moderate damage are provided in Appendix C.

[redacted] The distance corresponding to 1 psi peak overpressure (Section I, Chapter 2) should be used as the distance corresponding to 50 percent probability of light damage for all structures except Structure 11-2 (blast resistant structure) and Structures 11-14 through 11-24 (bridges). Light damage is not pertinent to blast resistance structures without windows. The distance corresponding to about 1 psi dynamic pressure (Section I, Chapter 2) should be used as the distance corresponding to 50 percent probability of light damage to bridges.

11-5 Shallow Underground Bursts [redacted]

[redacted] For very shallow underground bursts, air blast, as opposed to ground shock, is still the primary cause of damage to surface structures. For a given weapon yield, as the depth of burst increases, the proportion of the energy released to the atmosphere decreases. Therefore, the distance at which a given pressure level occurs on the surface, and consequently the damage radius for a particular type of structure, decreases. The damage distances for surface bursts as given in Figures 11-2 through 11-23 may be used also for shallow underground bursts if the surface burst distance is reduced for depth of burst in accordance with Figure 11-24.

11-6 Ground Shock and Cratering [redacted]

[redacted] The air blast from surface bursts or from underground bursts buried with a depth of cover of less than $35W^{1/3}$ feet (for W in kt) may cause severe damage to surface structures at ranges where damage from ground shock and cratering may be insignificant. Where the depth of burst is greater than $35W^{1/3}$ feet, ground shock may become the controlling damage producing mechanism. The peak ground shock motions that may be expected from surface and subsurface bursts may be inferred from the material presented in Section III of Chapter 2, by using the expressions therein for computing values of maximum displacement, velocity, and acceleration of the ground. The effects of the ground motions on structures and equipment may be computed from response spectra which are developed from the free-field ground motions, as described in Section II of this chapter for underground structures.

[redacted]

Table 11-5. [redacted] Factors for Obtaining Distances Corresponding to 50 Percent Probability of Moderate Damage to the Indicated Structure Type [redacted]

Structure Type	Moderate Damage Factor for Indicated Yield													
	30 Mt	10 Mt	3 Mt	1 Mt	300 kt	100 kt	30 kt	10 kt	3 kt	1 kt	0.3 kt	0.1 kt	0.03 kt	0.01 kt
11- 2	1.2	1.2	1.2	1.2	1.2	1.2	1.3	1.3	1.3	1.3	–	–	–	–
11- 3	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.4	1.5	1.5	1.5	1.5	1.5	1.5
11- 4	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3
11- 5	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3
11- 6	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.3	1.4	1.5	1.5	1.5	1.5	1.5
11- 7	1.1	1.1	1.1	1.2	1.2	1.3	1.5	1.5	1.7	1.9	2.2	–	–	–
11- 8	1.1	1.1	1.1	1.2	1.2	1.3	1.3	1.5	1.6	1.6	1.7	–	–	–
11- 9	1.1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.5	1.5	1.5	–	–	–
11-10	1.1	1.2	1.2	1.3	1.4	1.5	1.5	–	–	–	–	–	–	–
11-11	1.1	1.2	1.2	1.2	1.3	1.4	1.5	1.6	1.6	1.6	–	–	–	–
11-12	1.1	1.1	1.2	1.2	1.3	1.3	1.3	1.4	–	–	–	–	–	–
11-13	1.1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.5	1.5	–	–	–	–
11-14	1.0	1.0	1.0	1.0	1.1	1.1	1.3	1.4	–	–	–	–	–	–
11-15	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.2	1.2	1.2	–	–	–	–
11-16	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.2	1.2	1.2	–	–	–	–
11-17	1.0	1.0	1.0	1.1	1.1	–	–	–	–	–	–	–	–	–
11-18	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.1	–	–	–	–	–	–
11-19	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.2	–	–	–
11-20	1.0	1.0	1.0	1.0	1.1	1.1	1.2	1.3	–	–	–	–	–	–
11-21	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.2	1.3	–	–	–
11-22	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.2	1.2
11-23	1.4	1.4	1.3	1.3	1.3	1.3	1.3	1.3	1.2	1.2	1.2	1.1	–	–

*[redacted] These factors are to be applied to the appropriate scaled distance for the structure type and yield as obtained from Figures 11-2 through 11-23 (see Problem 11-1).

Problem 11-1 Calculation of Damage to Aboveground Structures from an Air Burst

[redacted] Figures 11-2 through 11-23 are families of isodamage curves that show scaled distance at which severe damage is expected to occur to the various structures that are described in Tables 11-1 and 11-2 as a function of weapon yield and height of burst. Data for yields not indicated may be found by interpolation. Scaled distances for moderate damage to the same structures may be obtained by multiplying the scaled distances of Figures 11-2 through 11-23 by the factors shown in Table 11-5.

[redacted] *Scaling.* A scaled damage distance may be read directly from the appropriate figure by entering the figure with the scaled height of burst and the appropriate yield. The relationships between the scaled height of burst and actual height of burst and between the actual ground distance and the scaled ground distance are:

$$\frac{h}{h_s} = \frac{d}{d_s} = W^{1/3}$$

where h and d are the height of burst and distance from ground zero, respectively, for a yield of W kt; and h_s and d_s are the corresponding scaled height of burst and distance for use with Figures 11-2 through 11-23.

[redacted] *Example* [redacted]

Given: A 30 kt weapon burst at an altitude of 1,865 feet.

Find.

(a) The distance from ground zero at which there is a 50 percent probability of severe damage to a multistory (3 to 8 stories) reinforced concrete building with concrete walls and a small window area.

(b) The distance from ground zero at which there is a 50 percent probability of moderate damage to a multistory (up to 4 stories) wall bearing building of the monumental type.

Solution:

(a) The corresponding scaled height of burst is

$$h_s = \frac{h}{W^{1/3}} = \frac{1{,}865}{(30)^{1/3}} = 600 \text{ feet.}$$

From Figure 11-3, at a scaled height of burst of 600 feet and a yield of 30 kt, the scaled distance at which there is a 50 percent probability of severe damage to a multistory reinforced concrete building with concrete walls and a small window area is 1,280 feet.

(b) From Figure 11-5, at a scaled height of burst of 600 feet and a yield of 30 kt, the scaled distance at which there is a 50 percent probability of severe damage to a multistory wall bearing building of the monumental type is 1,415 feet. From Table 11-5, the factor to obtain the scaled distance for moderate damage to the same structure (structure type 11-5) for a 30 kt weapon is 1.2. The corresponding scaled distance for moderate damage is

$$d_s \text{ (moderate)} = 1.2 \times d_s = 1.2 \times 1.415 = 1{,}700 \text{ feet.}$$

Answer:

(a) The actual ground distance for a 30 kt weapon is

$$d = d_s \times (W)^{1/3} = 1{,}280 \times (30)^{1/3} = 3{,}980 \text{ feet.}$$

(b) The actual ground distance for a 30 kt weapon is

$$d \text{ (moderate)} = d_s \text{ (moderate)} \times (W)^{1/3} = 1{,}700 \times (30)^{1/3} = 5{,}280 \text{ feet.}$$

[redacted]

Reliability: Since both the free-field overpressure and dynamic pressure pulses were assumed to be ideal, and since idealized loading functions were used in constructing Figures 11-2 through 11-23, no precise estimate of the reliability can be made for a specific situation. The ratio of the distance at which severe damage will occur to a particular structure to the distance at which moderate damage will occur to the same structure varies somewhat with height of burst. Therefore, use of the factors from Table 11-5 to obtain moderate damage distances is an approximation. A better estimate of the distance at which moderate damage will occur to a particular structure may be obtained by the methods described in Appendix C.

Related Material: See paragraphs 11-2 through 11-4, Tables 11-1 and 11-5, and Figure 11-1. See also Section I, Chapter 2, and Appendix C.

[redacted]

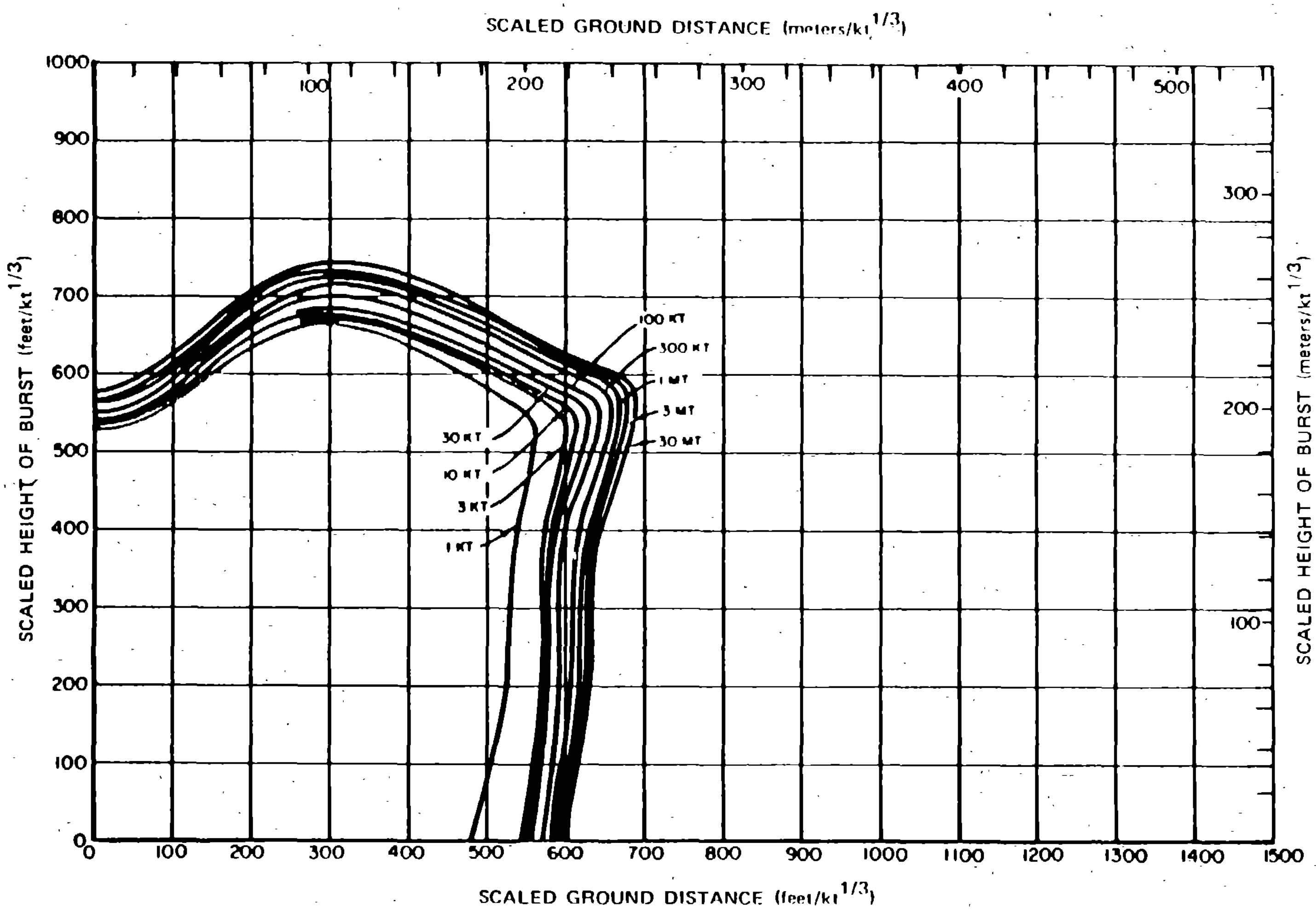

Figure 11-2. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Blast-Resistant Reinforced Concrete Buildings [redacted]

[redacted]

[redacted]

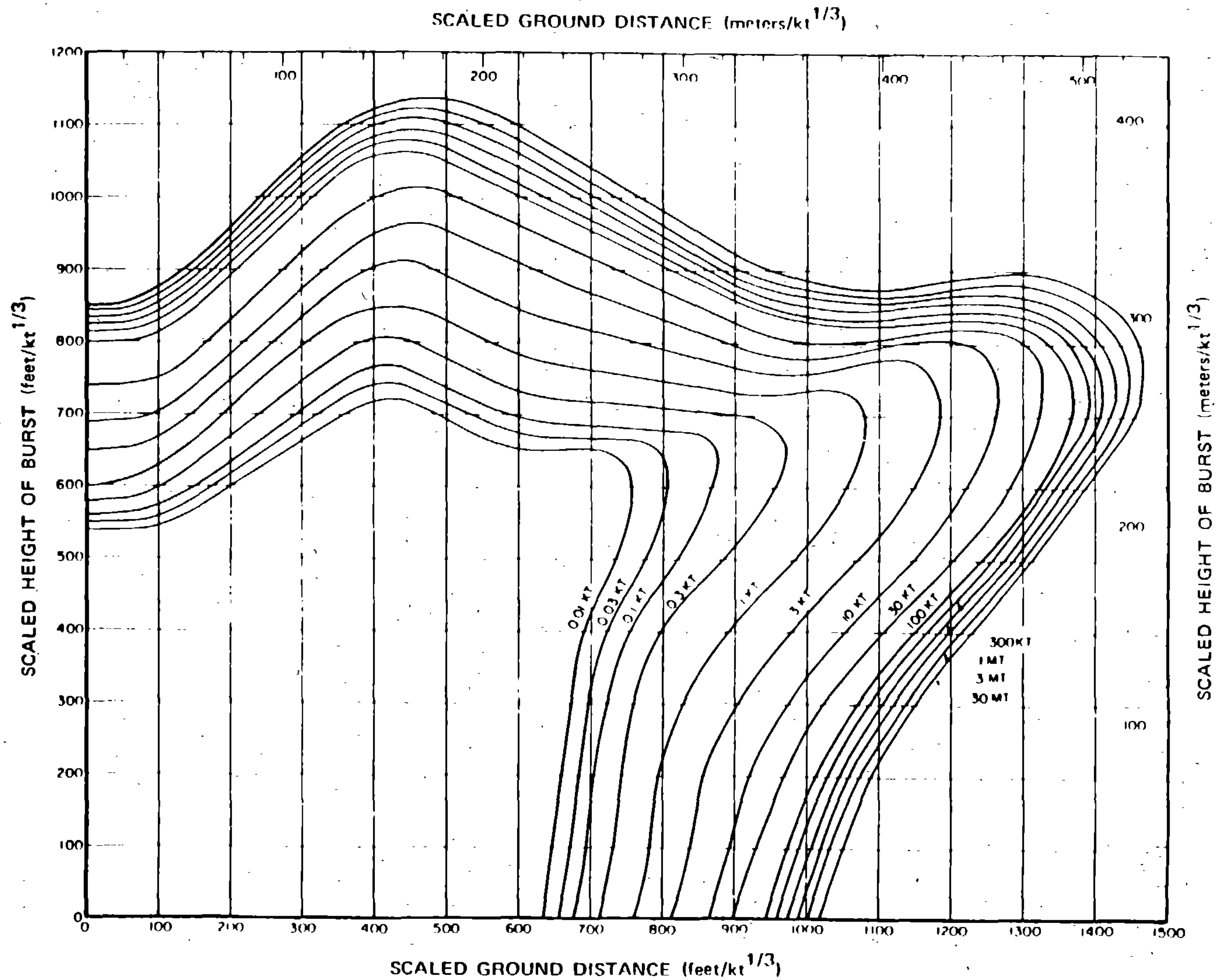

Figure 11-3. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Reinforced Concrete Buildings with Concrete Walls, Small Window Area, Three to Eight Stories [redacted]

[redacted]

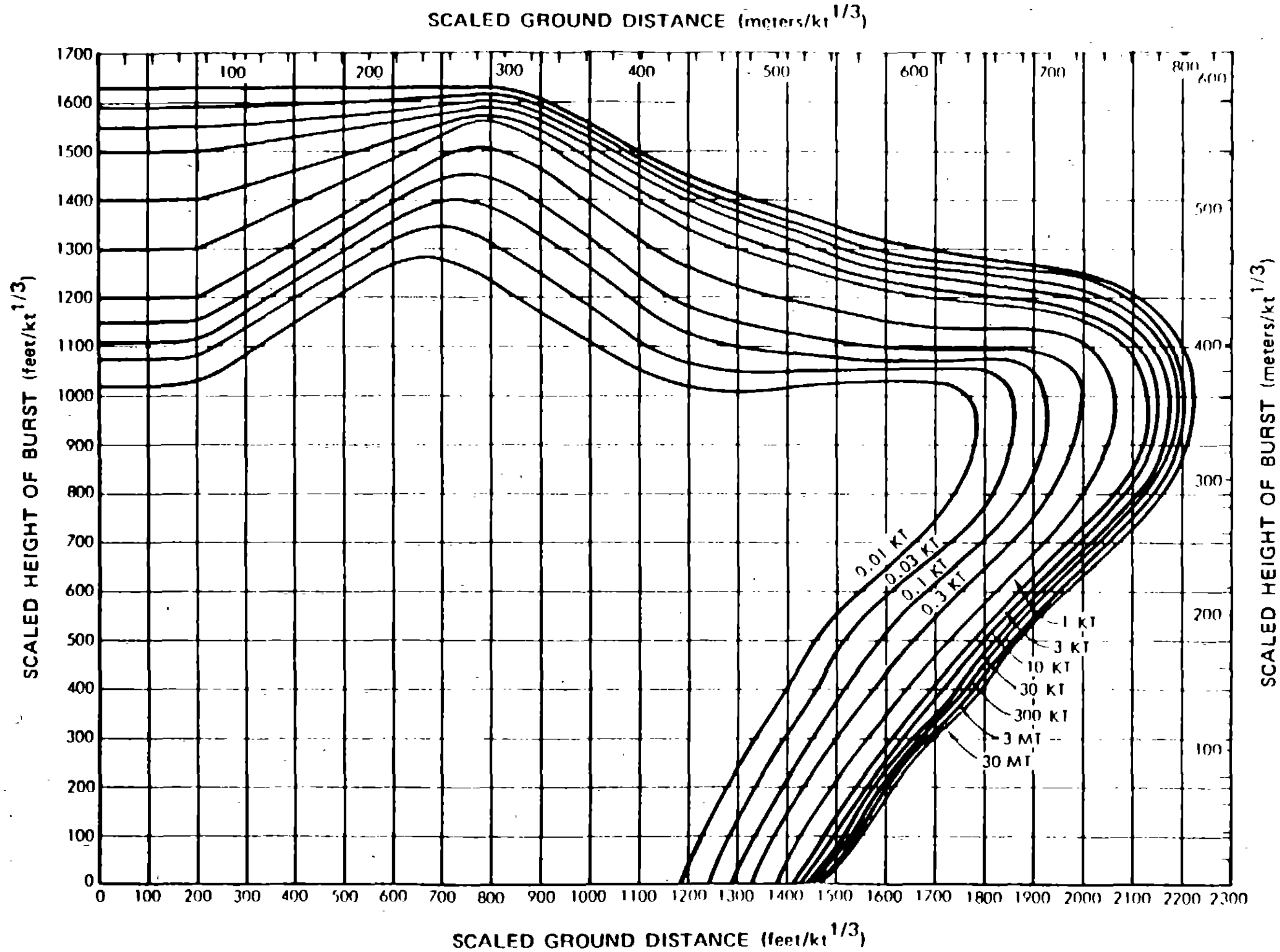

Figure 11-4. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Wall Bearing Buildings, Brick Apartment House Type, up to Three Stories [redacted]

[redacted]

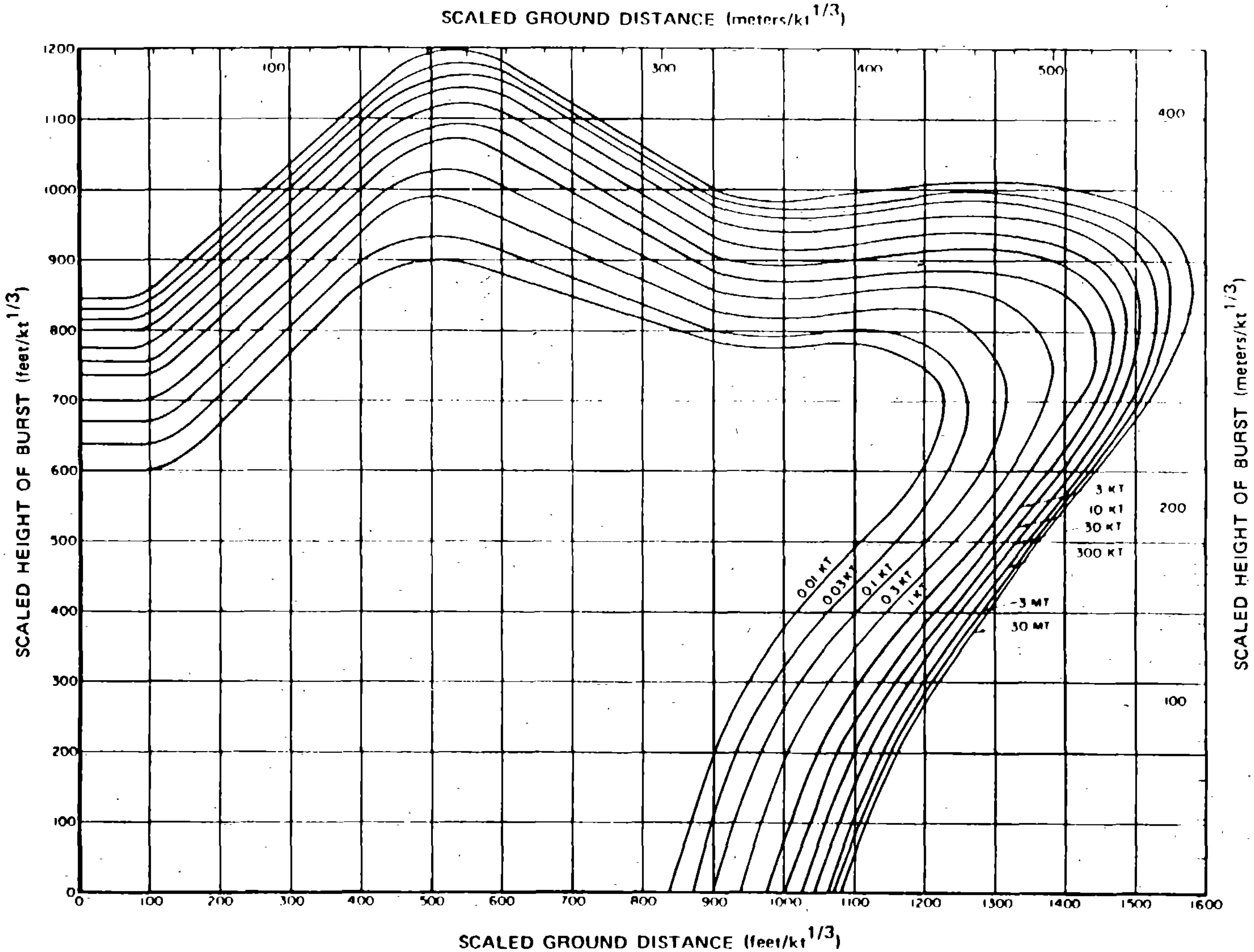

Figure 11-5. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Wall Bearing Building, Monumental Type, up to Four Stories [redacted]

[redacted]

[redacted]

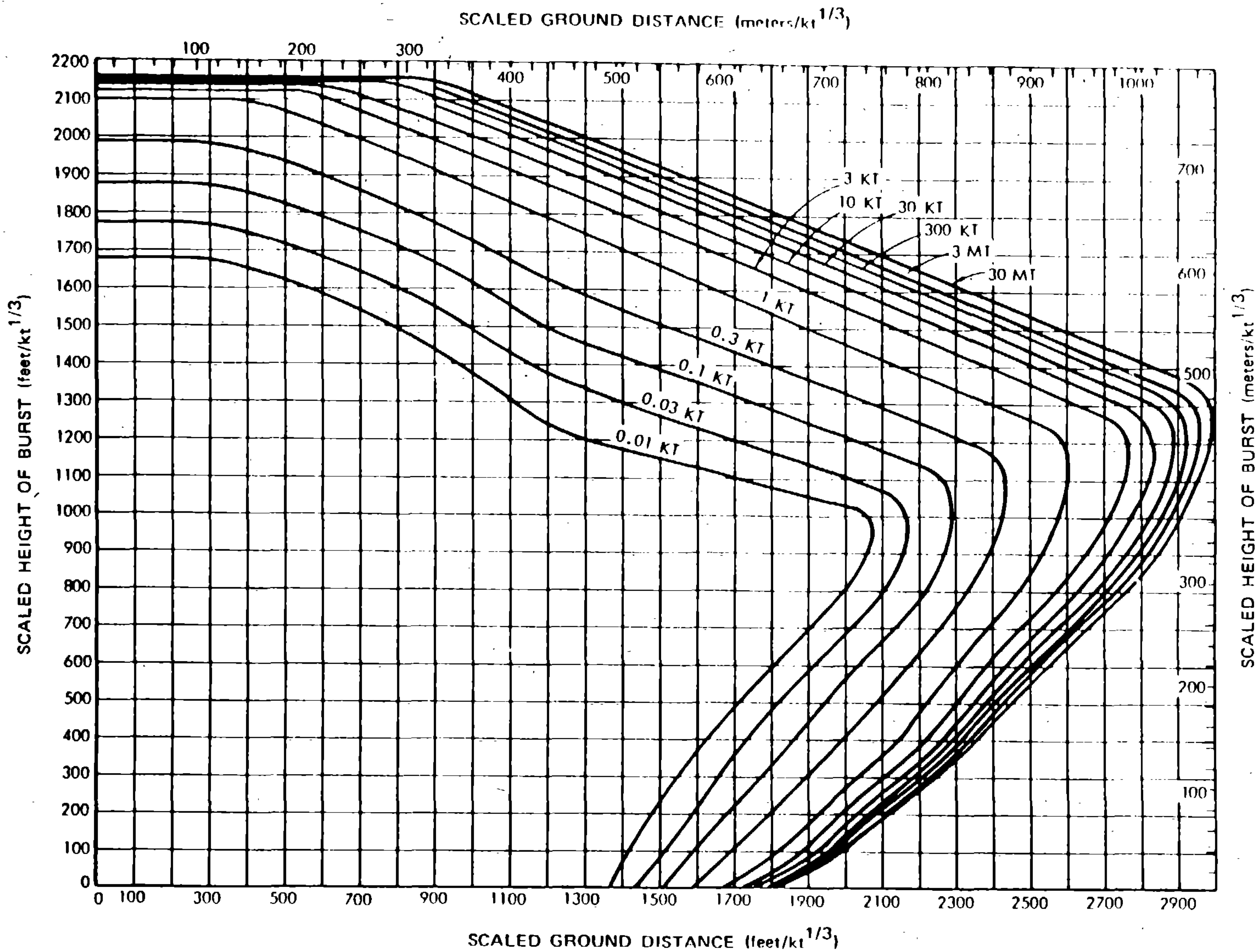

Figure 11-6. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Wood Frame Buildings, House Type, One or Two Stories [redacted]

[redacted]

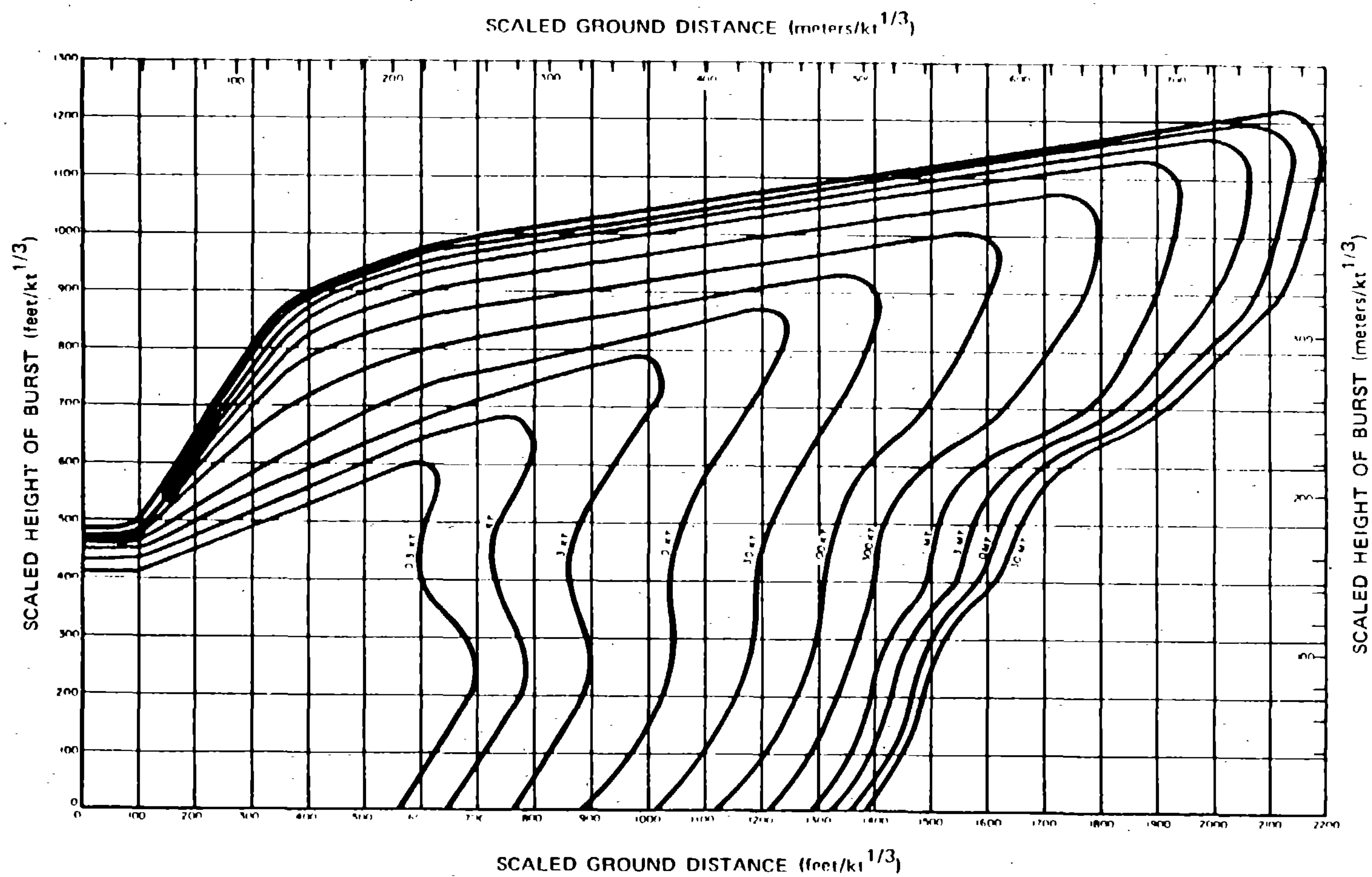

Figure 11-7. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Light Steel Frame Industrial Buildings, Single Story, Frangible Walls, up to 5 Ton Crane Capacity [redacted]

[redacted]

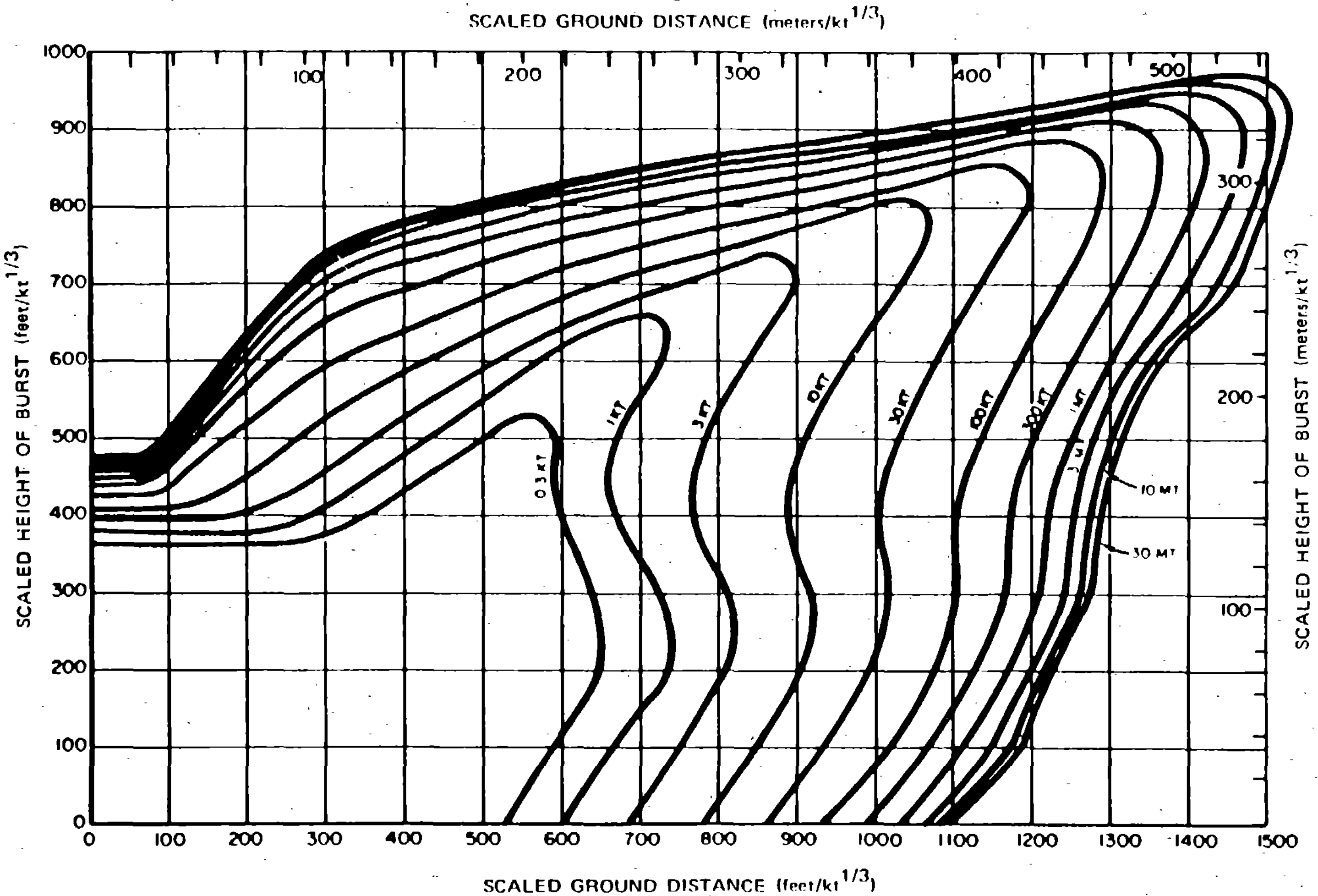

Figure 11-8. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Heavy Steel Frame Industrial Buildings, Single Story, Frangible Walls, 25 to 50 Ton Crane Capacity [redacted]

[redacted]

[redacted]

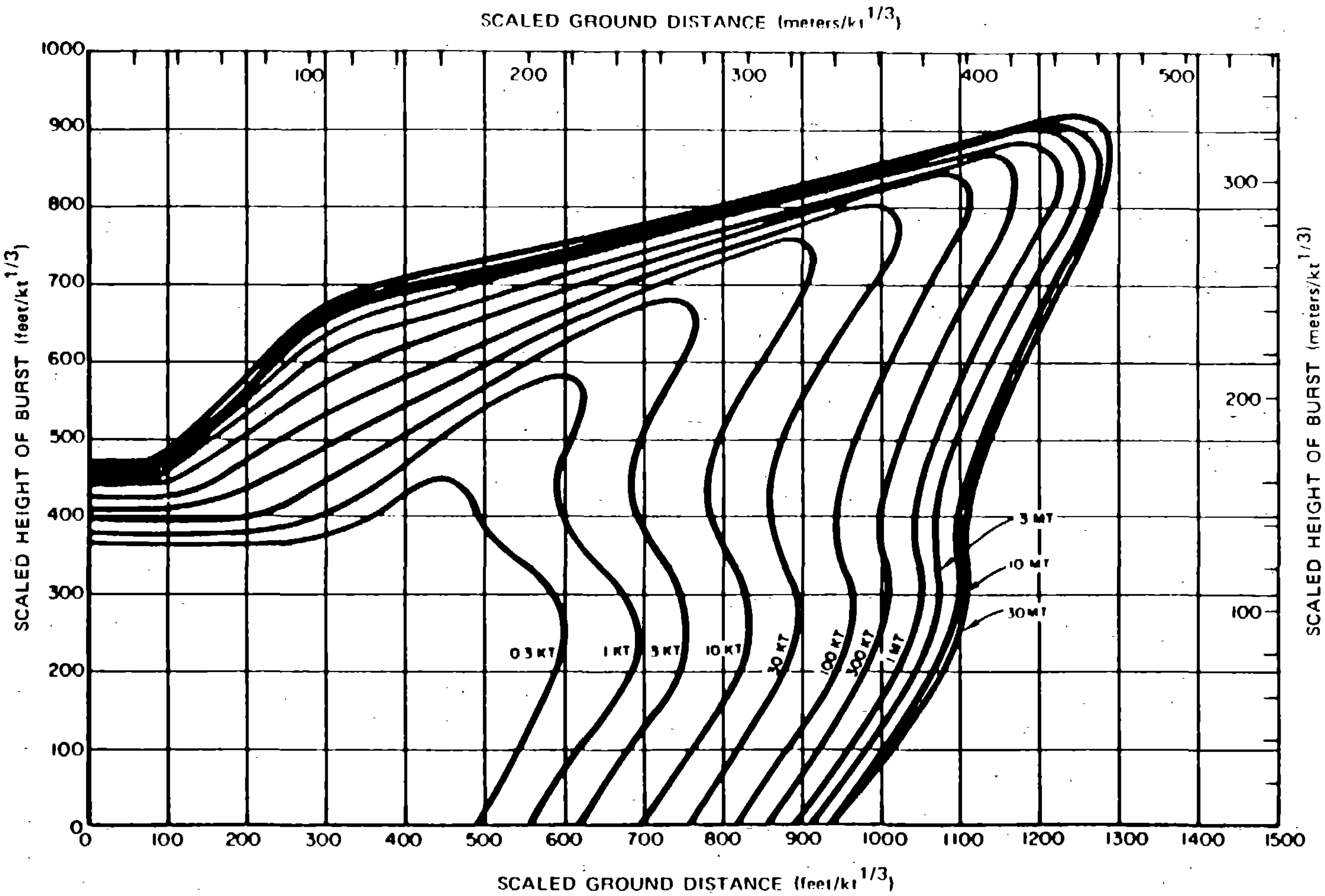

Figure 11-9. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Heavy Steel Frame Industrial Buildings, Single Story, Frangible Walls, 60 to 100 Ton Crane Capacity [redacted]

[redacted]

[redacted]

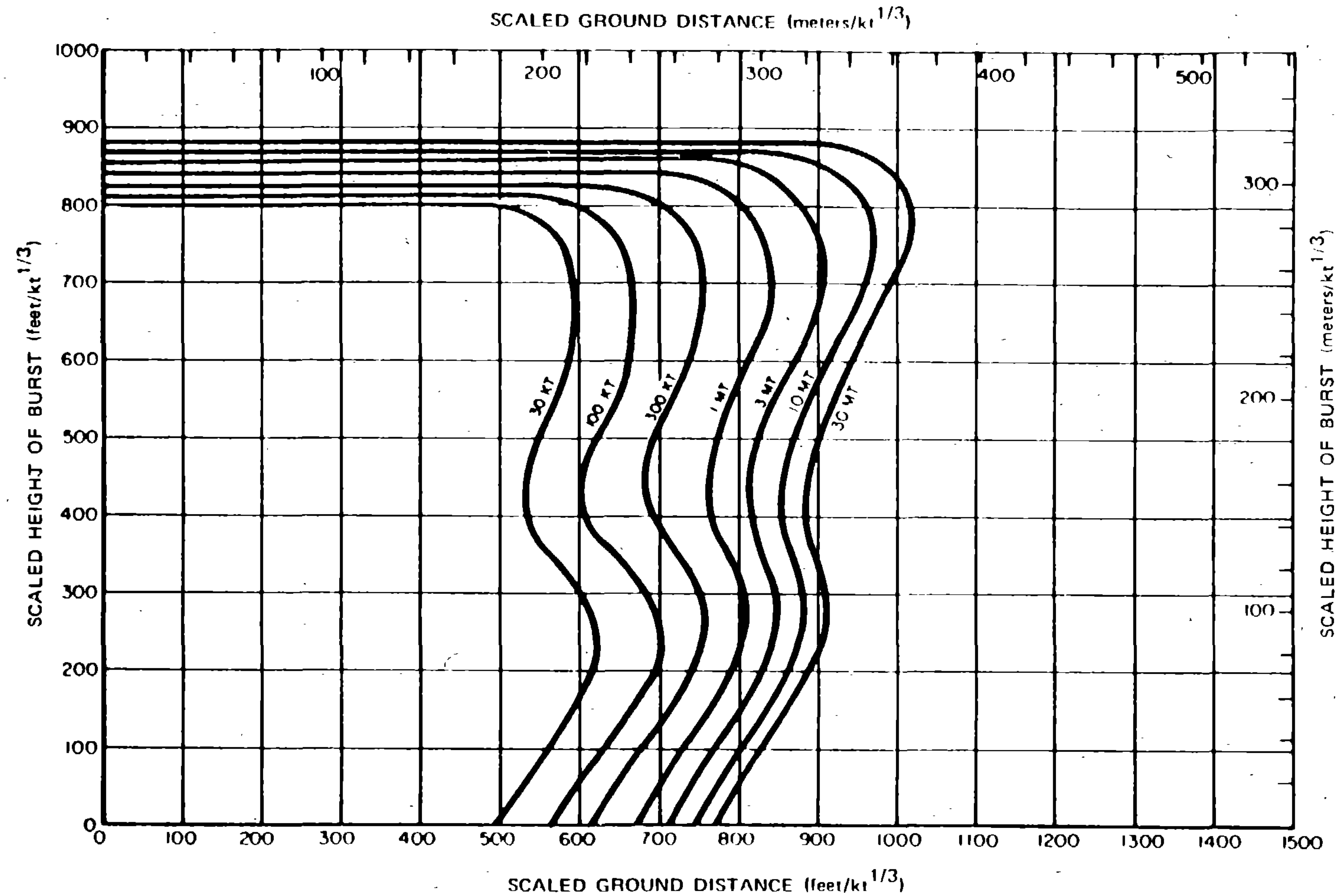

Figure 11-10. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Steel Frame Office Type Buildings, 3- to 10-Stories, Frangible Walls, Earthquake Resistant Construction [redacted]

[redacted]

[redacted]

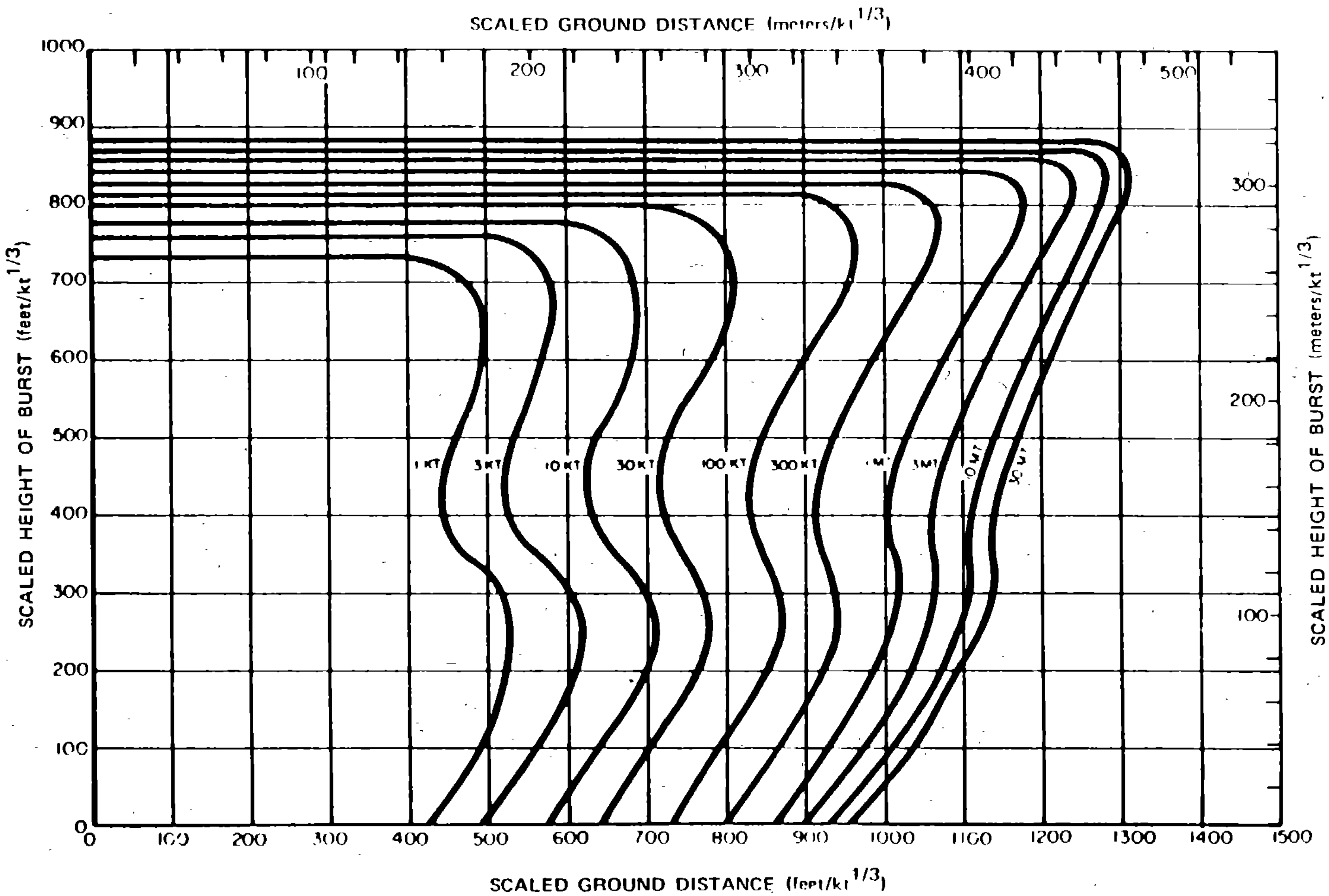

Figure 11-11. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Steel Frame Office Type Buildings, 3- to 10-Stories, Frangible Walls, Non-Earthquake Resistant Construction [redacted]

[redacted]

[redacted]

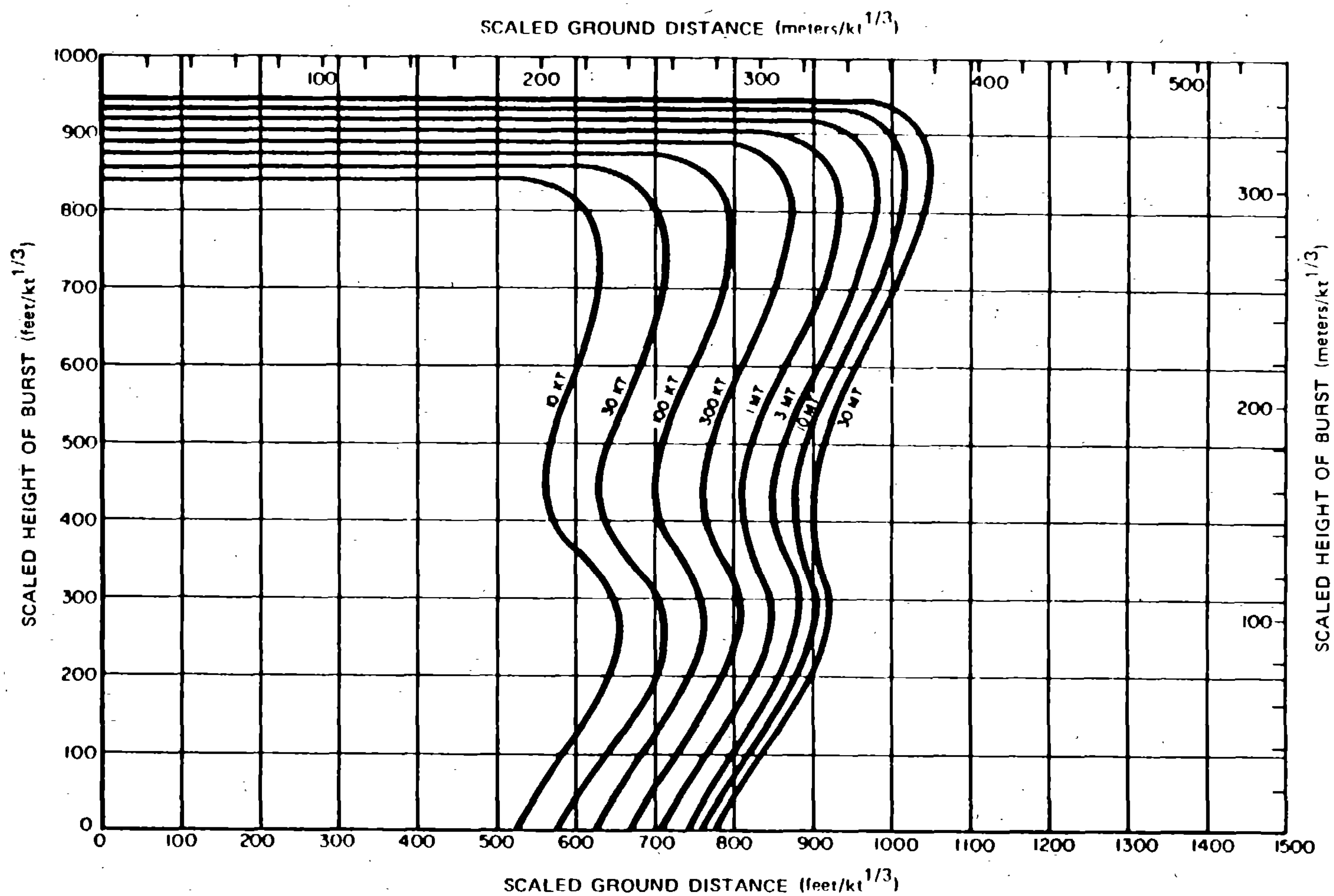

Figure 11-12. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Reinforced Concrete Frame Office Type Buildings, 3- to 10-Stories, Frangible Walls, Earthquake Resistant Construction [redacted]

[redacted]

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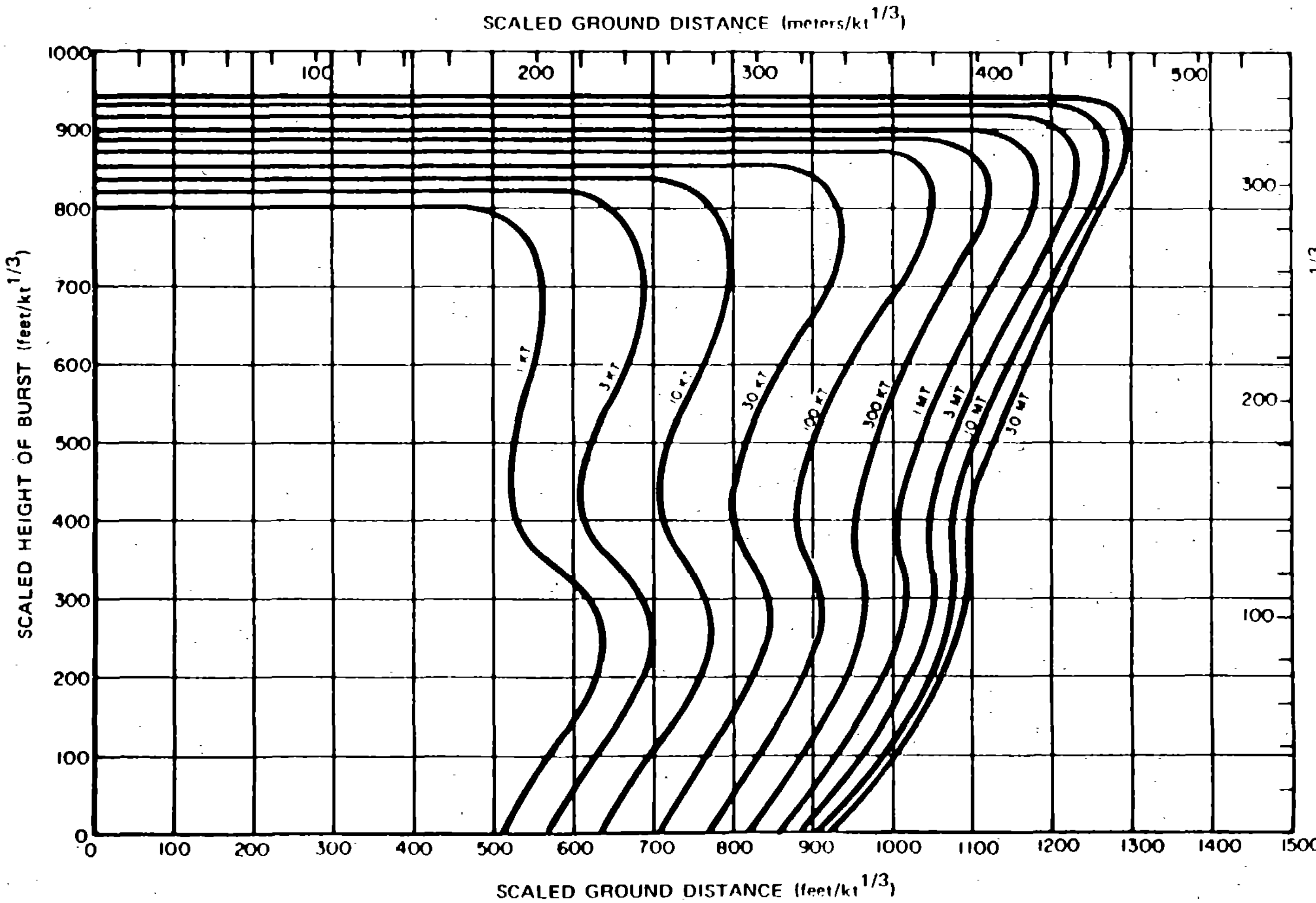

Figure 11-13. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Multistory Reinforced Concrete Frame Office Type Buildings, 3- to 10-Stories, Frangible Walls, Non-Earthquake Resistant Construction [redacted]

[redacted]

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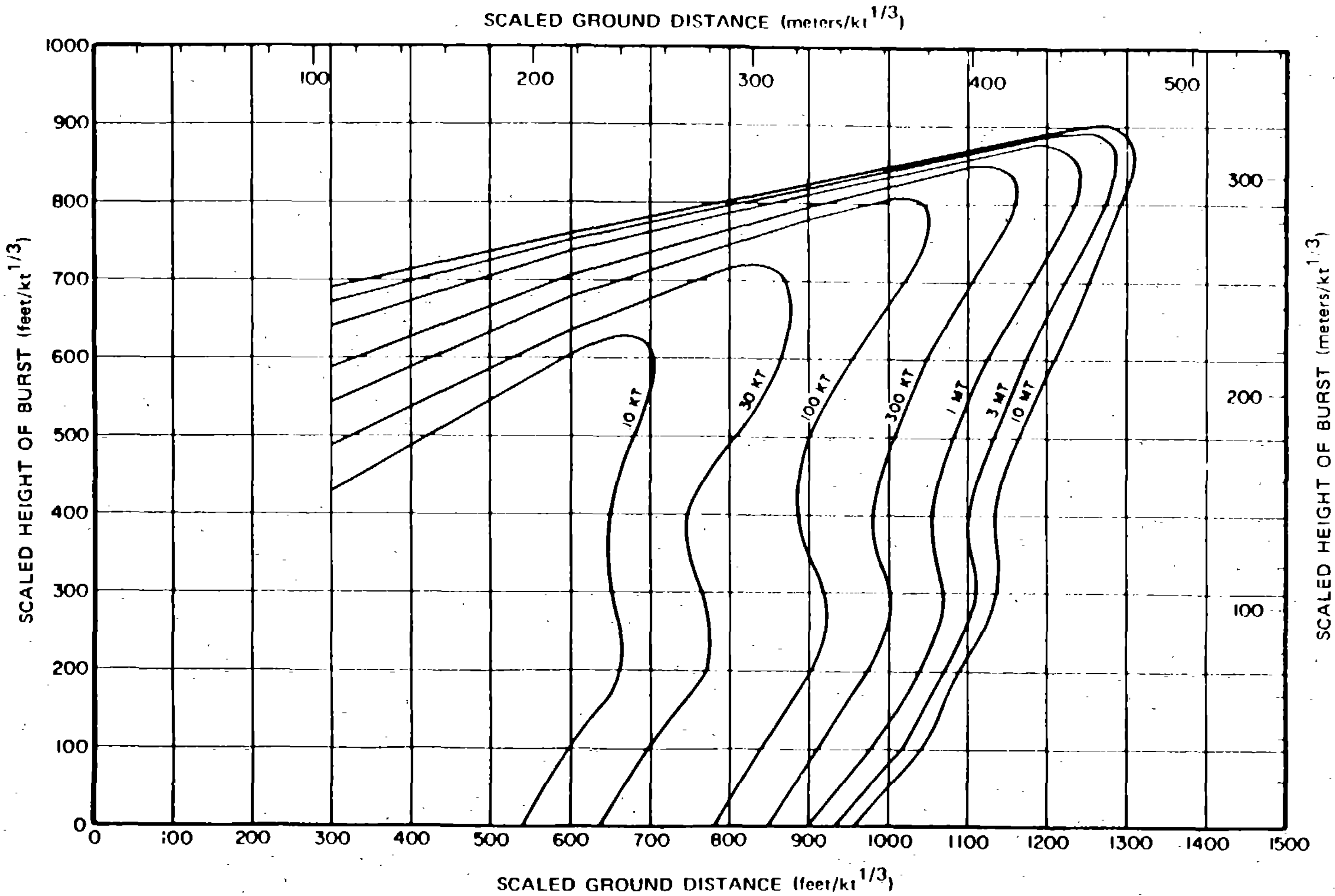

Figure 11-14. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Four-Lane Highway Truss Bridges and Double Track Ballast Floor Railroad Truss Bridges, Span 200 ft to 400 ft [redacted]

[redacted]

[redacted]

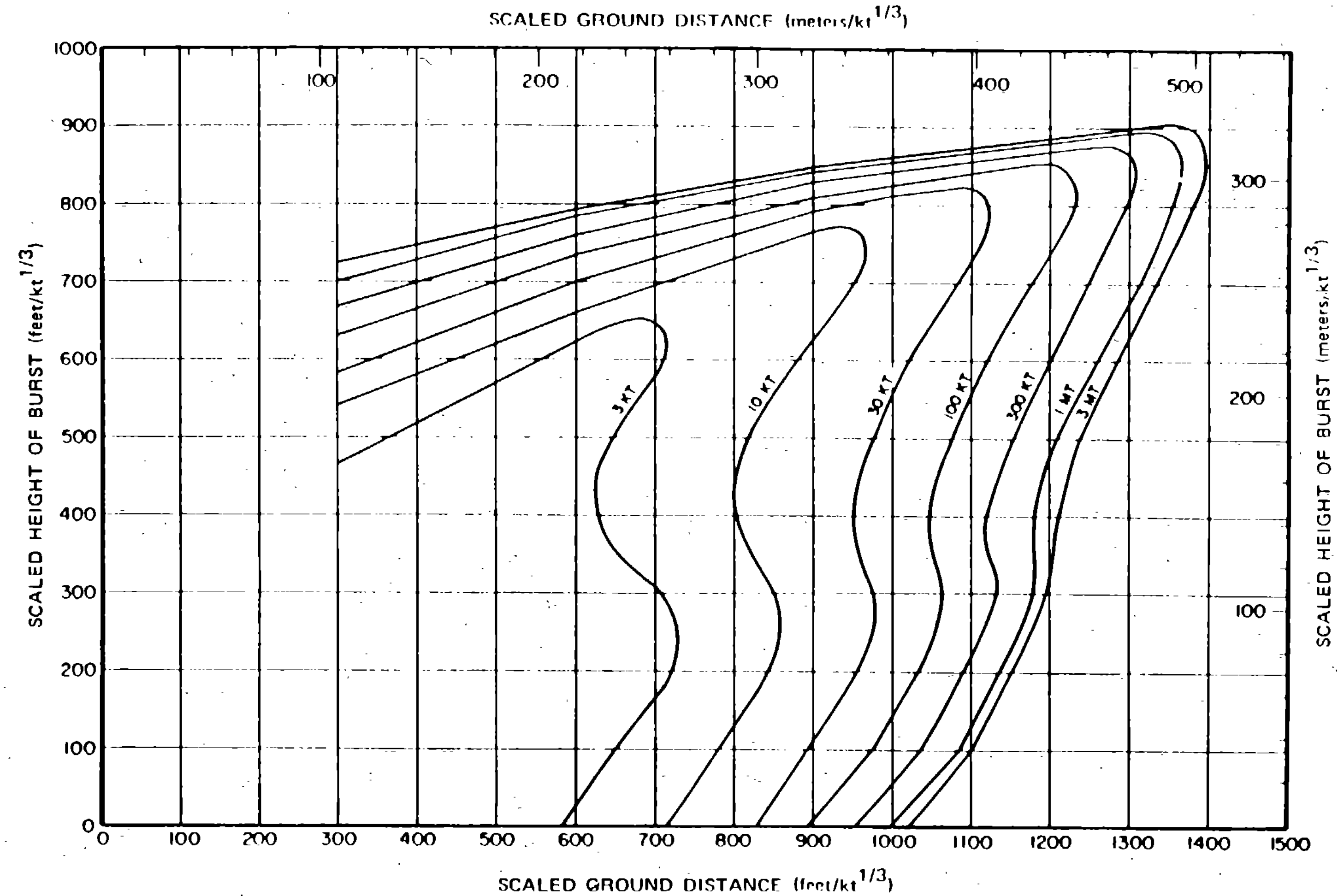

Figure 11-15. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Two-Lane Highway Truss Bridges, Spans 200 ft to 400 ft; Single Track Ballast Floors and Double Track Open Floor Railroad Truss Bridges, Spans 200 ft to 400 ft; and Single Track Open Floor Railroad Truss Bridges, Span 400 ft [redacted]

[redacted]

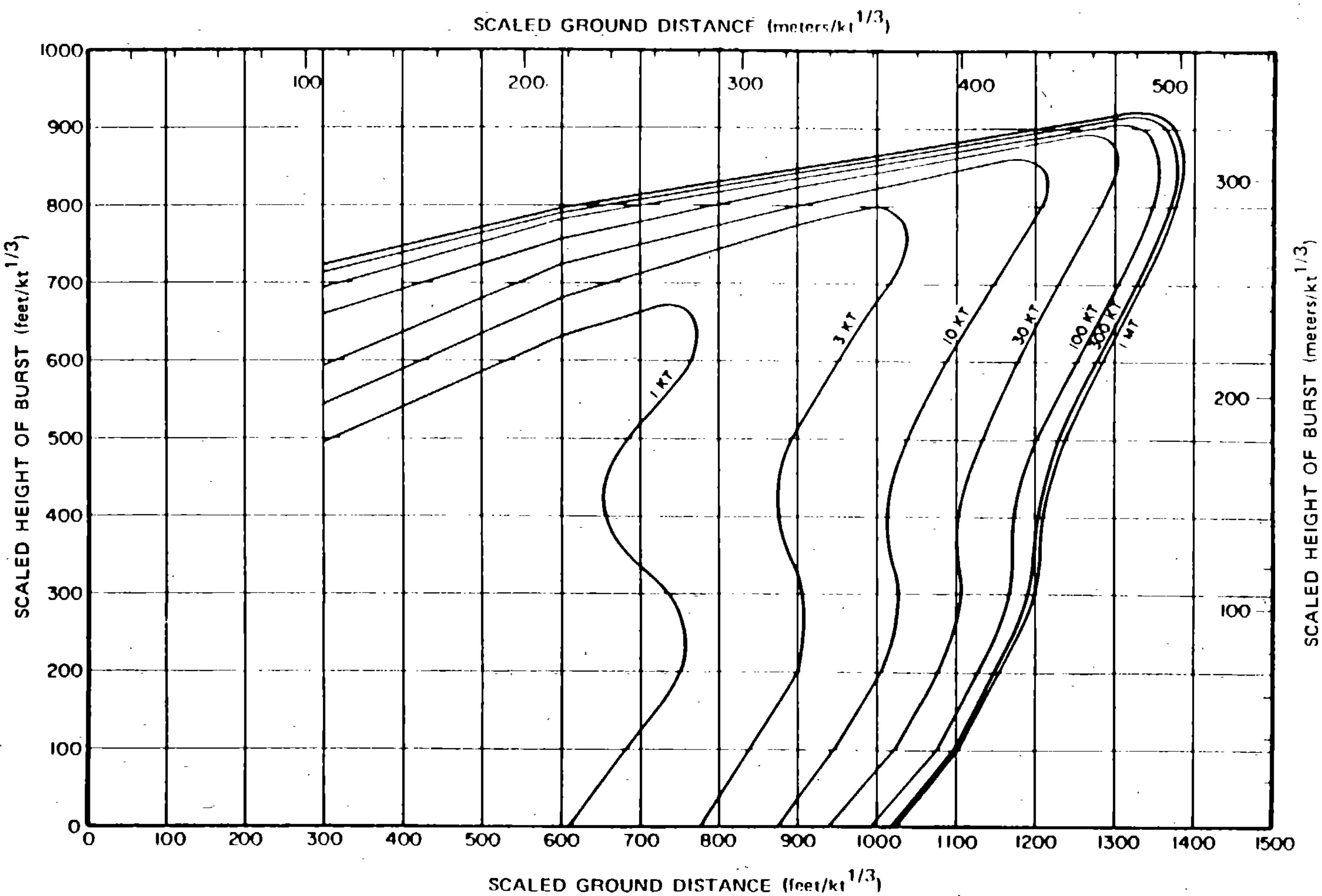

Figure 11-16. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Single Track Open Floor Railroad Truss Bridges, Span 200 ft [redacted]

[redacted]

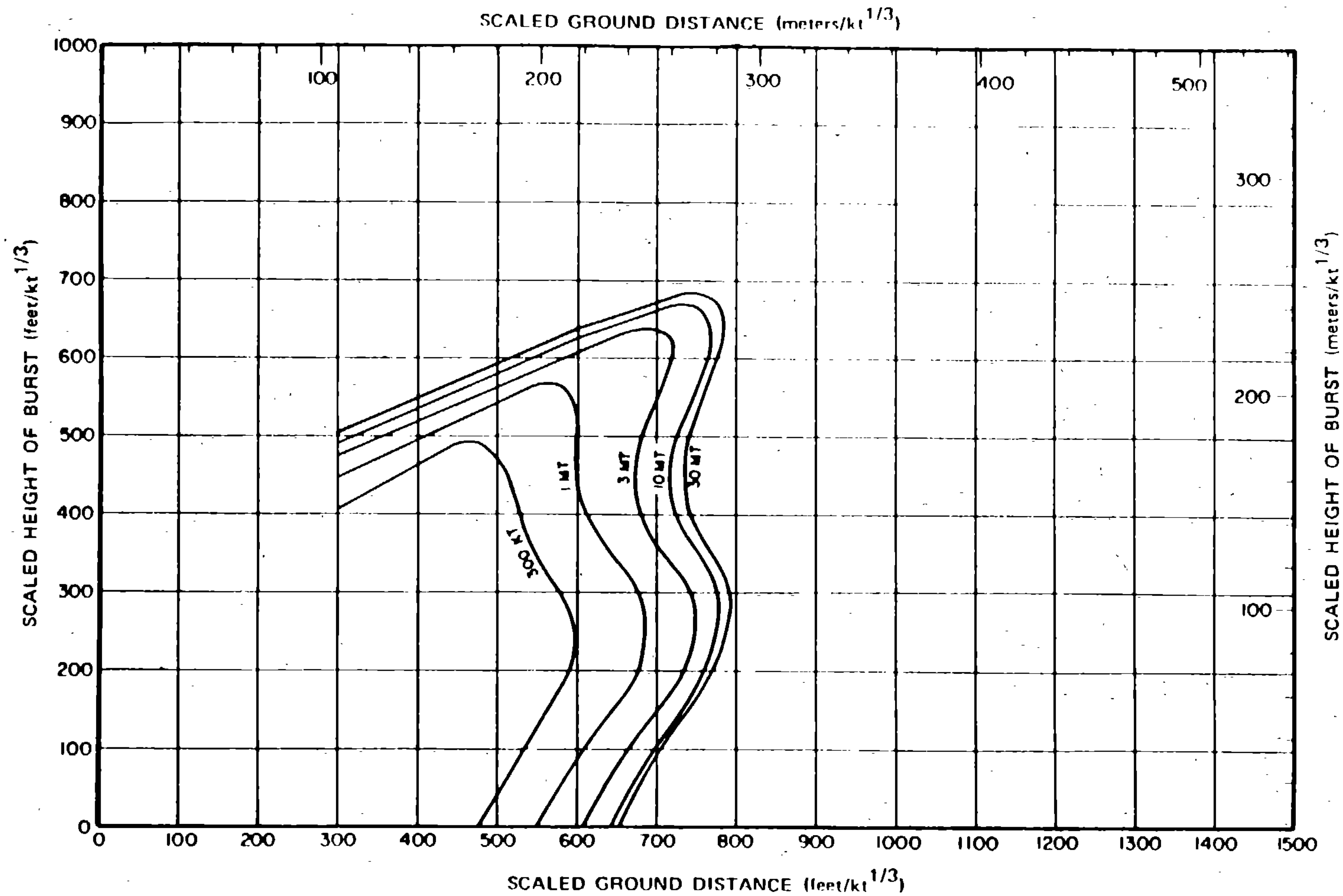

Figure 11-17. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Four-Lane Through Highway Girder Bridges, Spans 75 ft [redacted]

[redacted]

[redacted]

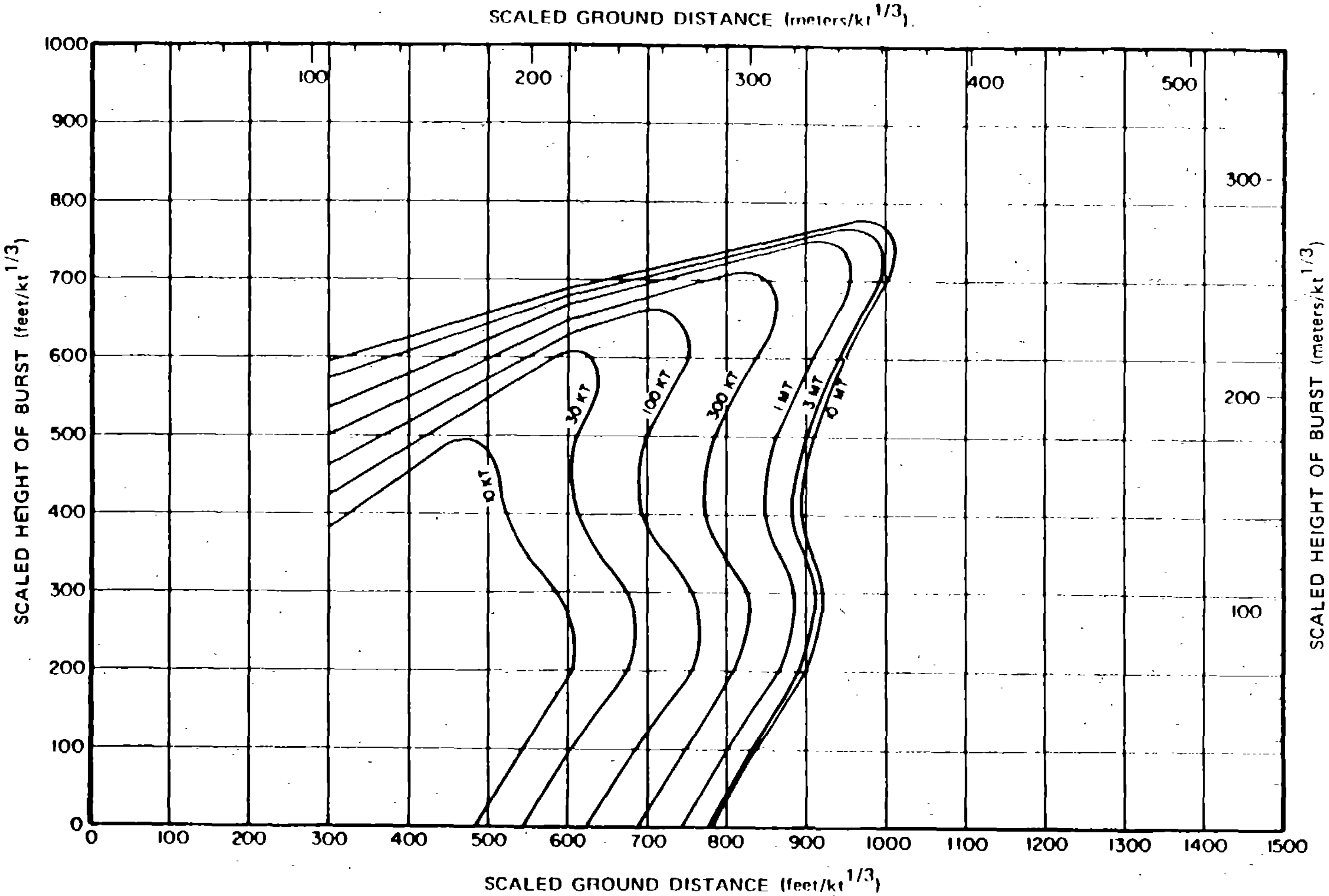

Figure 11-18. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Two-Lane Deck, Two-Lane Through, and Four-Lane Deck Highway Bridges, Spans 75 ft; Double Track Deck, Open or Ballast Floor Railroad Girder Bridges, Span 75 ft; Single or Double Track Through, Ballast Floor Railroad Girder Bridges, Spans 75 ft [redacted]

[redacted]

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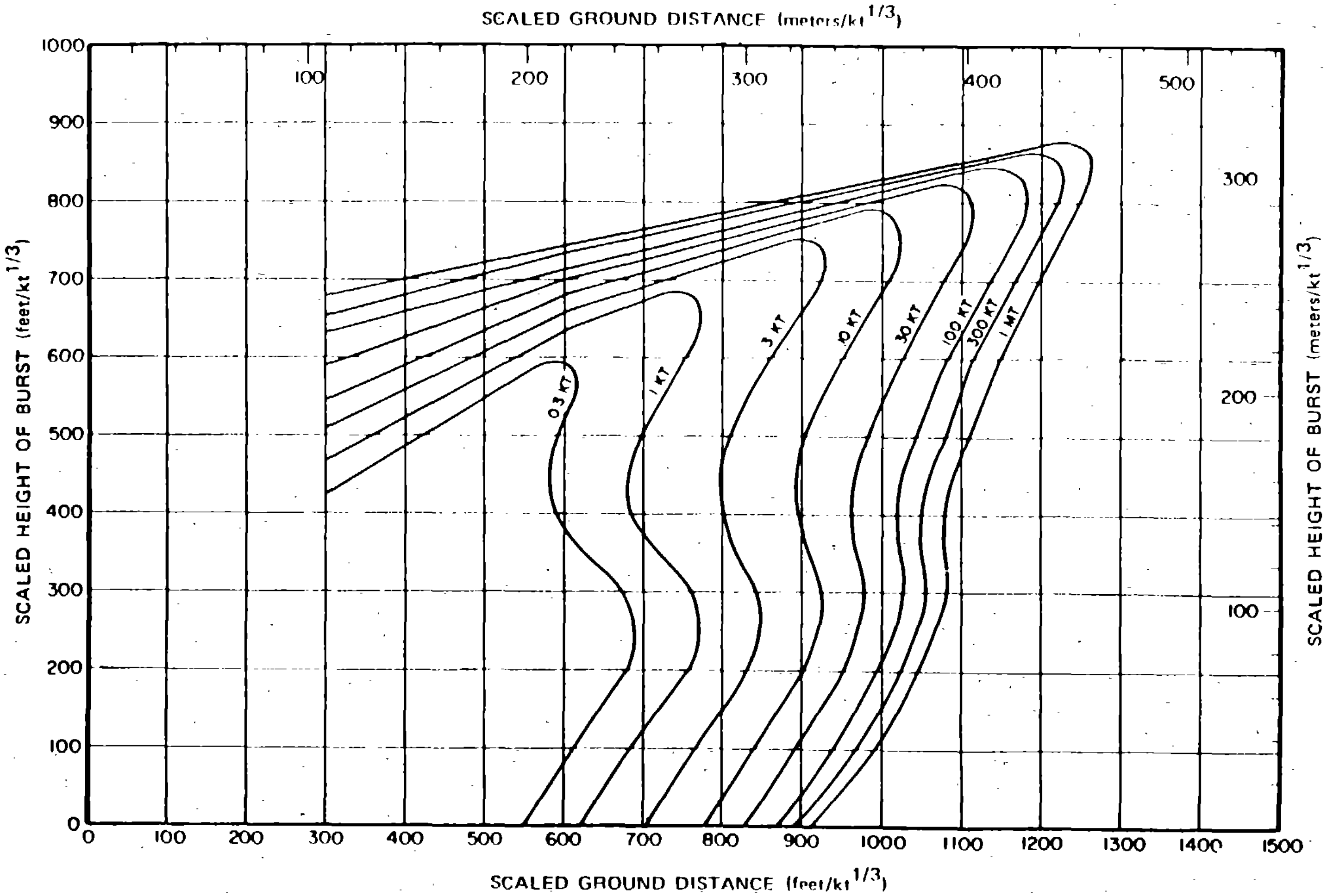

Figure 11-19. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Single Track Deck, Open or Ballast Floor, and Single or Double Track Through, Open Floor Railroad Girder Bridges, Spans 75 ft [redacted]

[redacted]

[redacted]

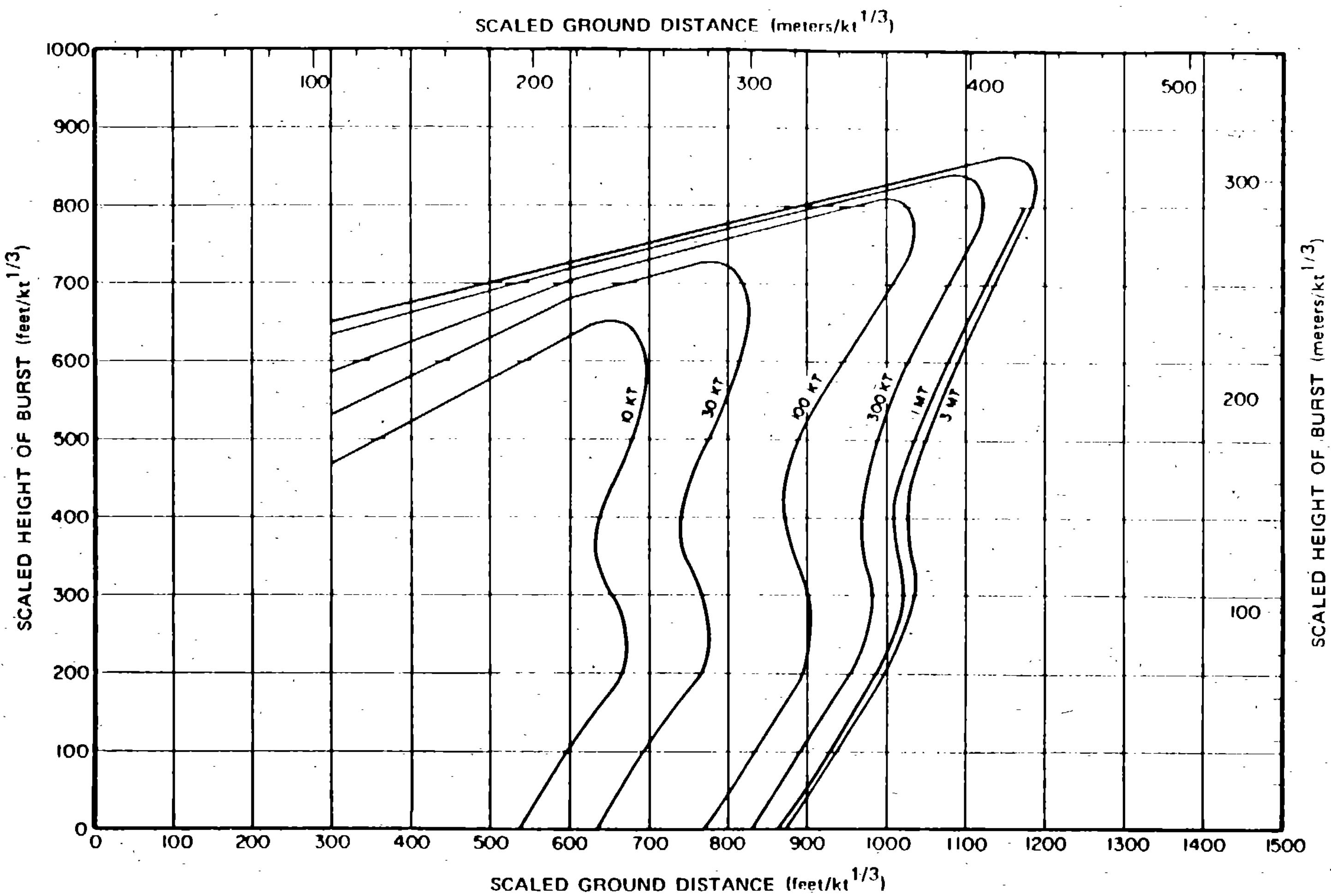

Figure 11-20. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Two-Lane Through and Four-Lane Deck or Through Highway Girder Bridges, Spans 200 ft; Double Track Deck or Through, Ballast Floor Railroad Girder Bridges, Spans 200 ft [redacted]

[redacted]

[redacted]

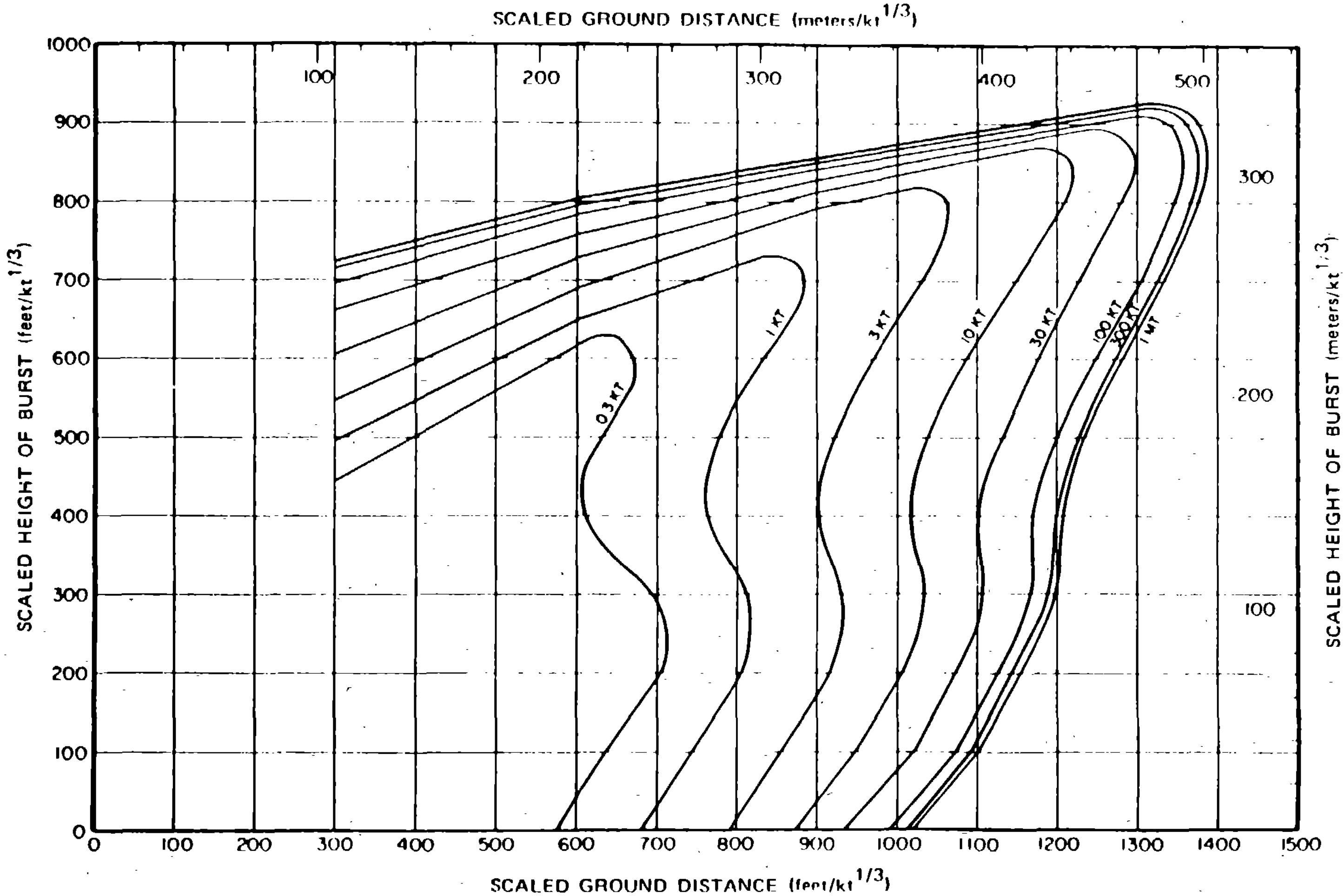

Figure 11-21. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Two-Lane Deck Highway Girder Bridges, Spans 200 ft; Single Track Deck or Through, Ballast Floor, and Double Track Deck or Through, Open Floor Railroad Girder Bridges, Spans 200 ft [redacted]

[redacted]

[redacted]

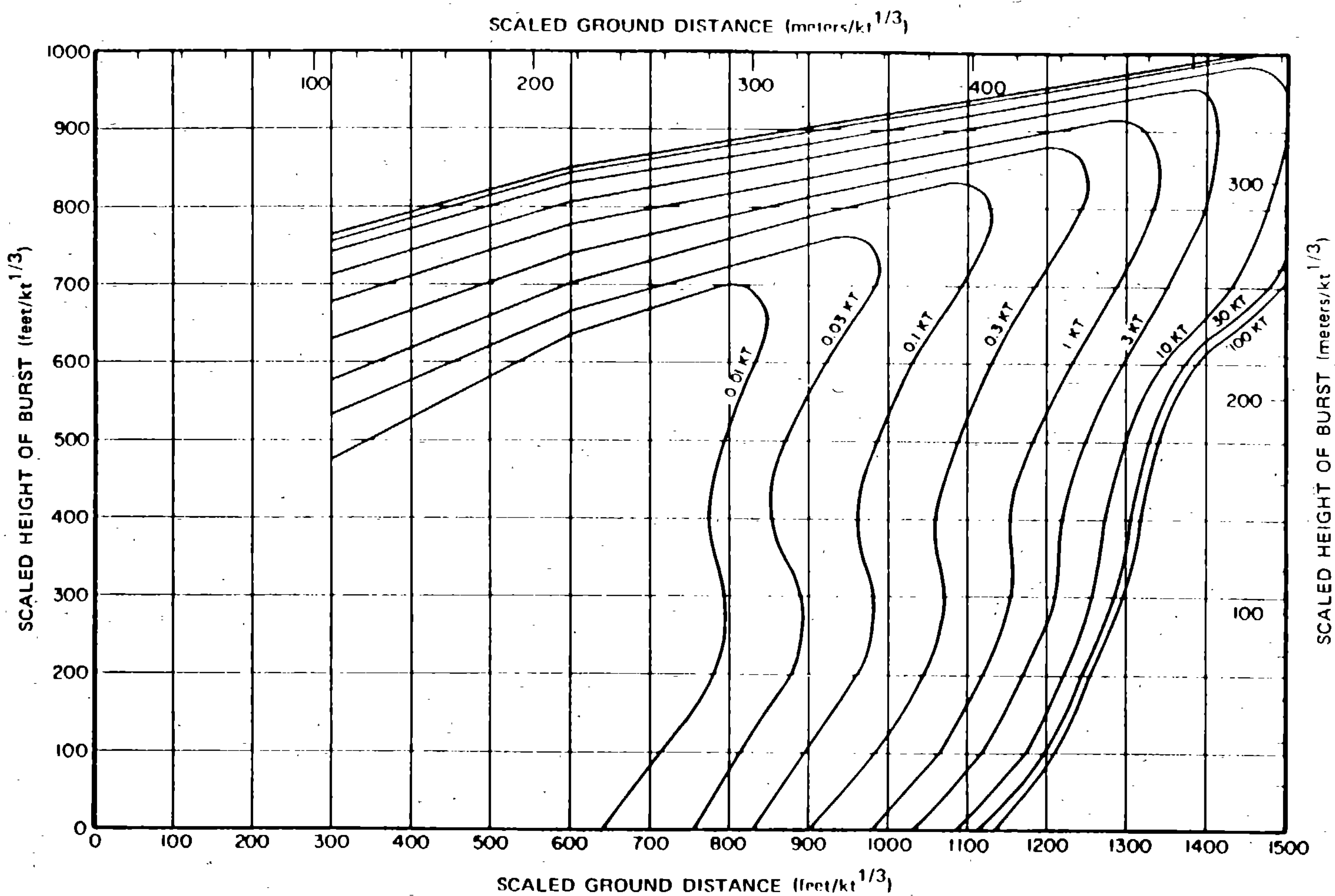

Figure 11-22. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage to Single Track Deck or Through, Open Floor, Railroad Girder Bridges, Spans 200 ft [redacted]

[redacted]

[redacted]

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Figure 11-23. [redacted] Isodamage Curves for Fifty Percent Probability of Severe Damage [redacted] to Floating Bridges, U.S. Army Standard M-2 and M-4, Random Orientation

[redacted]

Problem 11-2 Calculation of Damage-Distance Reduction for Surface Targets from Shallow Underground Bursts

[redacted] The distance at which damage occurs to surface targets decreases for shallow underground bursts when compared to the damage distance for surface bursts, even though air blast remains the primary damage mechanism. The amount of reduction of the damage distance may be obtained from Figure 11-24, which presents data for a shallow subsurface 1 kt burst. The distance reduction obtained from Figure 11-24 is applicable to the isodamage curves of Figures 11-2 through 11-23. The reductions also may be applied to damage distances for POL storage tanks (Figures 11-52 through 11-55) as well as to the air blast damage distances for parked aircraft (Chapter 13), military field equipment (Chapter 14), and forest stands (Chapter 15). The distance reductions derived from Figure 11-24 are applied to the damage distance for a surface burst of the desired yield and the appropriate target.

[redacted] *Scaling.* For yields other than 1 kt, scale as follows:

$$\frac{d_r}{d_1} = \frac{h}{h_1} = W'^{1/3}$$

where d_1 and h_1 are the distance reduction and depth of burst, respectively, for 1 kt; and d_r and h are the corresponding distance reduction and depth of burst for a yield of W' kt.

[redacted] *Example* [redacted]

Given: A 125 kt burst at a 50 foot depth.

Find: The distance at which there is a 50 percent probability of severe damage to a single story heavy steel frame industrial building with frangible walls and a 40 ton crane capacity.

Solution: The corresponding depth of burst for 1 kt is

$$h_1 = \frac{h}{W'^{1/3}} = \frac{50}{(125)^{1/3}} = 10 \text{ feet.}$$

From Figure 11-24, the damage distance reduction produced by exploding a 1 kt weapon 10 feet underground is 90 feet. From Figure 11-8, the scaled ground distance for severe damage to this structure from a 125 kt surface burst is 940 feet. The actual damage distance for a 125 kt surface burst is

$$d = d_s \times W'^{1/3} = 940 \times (125)^{1/3}$$

$$= 4{,}700 \text{ feet.}$$

Answer: The corresponding distance reduction for a 125 kt weapon is

$$d_r = d_1 \times W'^{1/3} = 90 \times (125)^{1/3}$$

$$= 450 \text{ feet.}$$

This value is subtracted from the surface burst damage-distance to obtain the corrected distance for the stated conditions

$$d = 4{,}700 - 450 = 4{,}250 \text{ feet.}$$

Reliability: Figure 11-24 is based on distance reduction for overpressures as obtained on full-scale field tests; however the damage distance for a given target is subject to the uncertainties described in Problem 11-1.

Related Material: See paragraph 11-5; also see Figures 11-2 through 11-23, paragraph 13-9, paragraphs 14-7 through 14-9, and Figures 15-1 through 15-34.

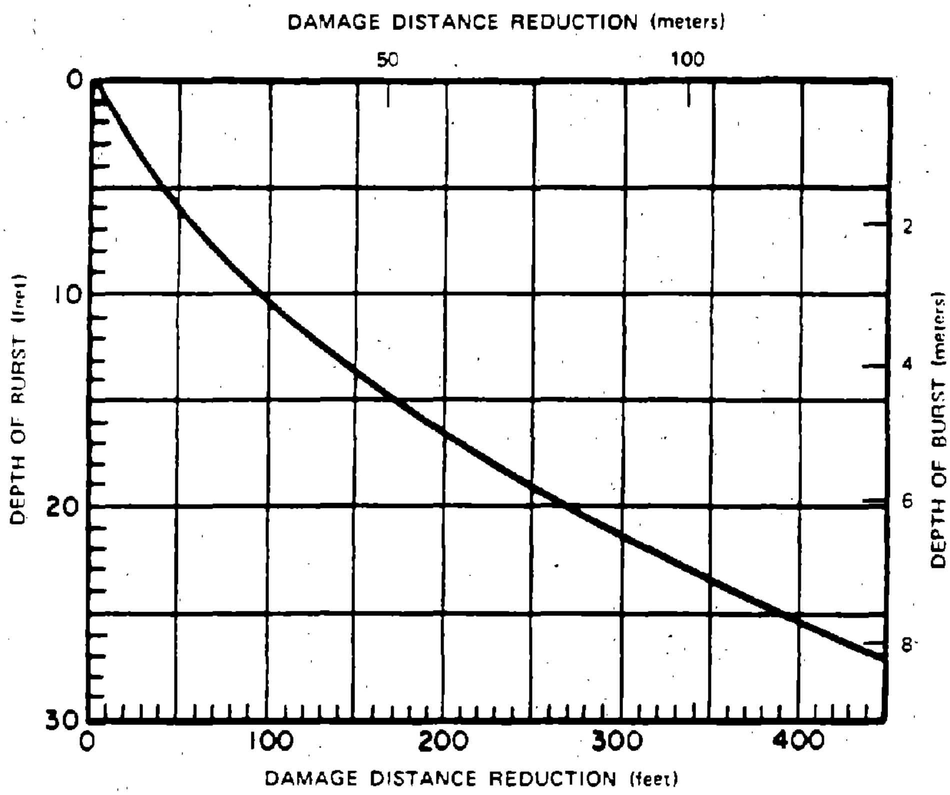


Figure 11-24. [redacted] Damage-Distance Reduction for Surface Structures from 1 kt Bursts in Soil [redacted]

SECTION II

[redacted] DAMAGE TO BELOWGROUND STRUCTURES [redacted]

[redacted] This section presents data for evaluation of the vulnerability of structures buried in soil and rock. The method of presentation differs considerably from that in the preceding section. This difference results principally from the fact that the structural types considered previously are conventional surface structures designed to resist conventional, natural loads, whereas structural types considered here are designed to resist the effects of nuclear weapons. Since the structures are designed intentionally for blast loads and normally for a particular purpose, it is difficult to categorize them for a general purpose or as having a characteristic resistance to loading which is rather narrowly banded as was done in Section I.

[redacted] The medium in which they are located can be a major distinguishing feature of protective structures. If they are buried in soil, the structure must provide the resistance to load completely. On the other hand, the rock surrounding a tunnel possesses an inherent resistance. Lining of the tunnel can confine the surrounding rock to augment the strength of the rock, or it can provide protection to the tunnel contents from the rock which has failed around the opening. Accordingly, this section is divided into two parts: structures buried in soil, and lined and unlined openings in rock. For the structures buried in soil, there is an added division depending upon whether the burial depth is shallow or deep. Several structural configurations are considered, with the primary breakdown being between shells, such as arches, domes and silos, and rectangular structures.

[redacted] Figures and tables that define the resistance of structural types are intended to present the median level of the capacity of the structure. That is, the structure is believed to have an equal chance of surviving or failing under the postulated conditions resulting from the analysis. For a protective system, a survival probability in excess of the 50 percent implied herein usually is required. Methods given in Appendix C may be used in conjunction with data given in this section to define any desired survival probability.

[redacted] Buried structures are located in a natural medium, which is subject to the normal vagaries of nature. The potential existence of unknown conditions within the soil produces ambiguities in the fundamental data that result in a relatively large scatter among the direct measurements. The data presented here have a generally wide experimental background as a foundation, but the variations within even a single experiment can be much wider than the variations that exist in tests of aboveground structures. Thus, the confidence levels for buried structures, as manifested by the corresponding standard deviations in Appendix C, are poorer than those presented for surface structures.

[redacted] Finally, the types and uses of buried structures can be extremely broad. As a result, a detailed method of analysis of selected structural configurations of a given resistance could be presented, or a general method of analysis for broad classes of structures could be provided. The general analysis of broad classes of structures has the advantage of being more complete, but simultaneously it is more complex. However, because detailed examples are presented for each of the broad classes, the problem of complexity has been largely circumvented in the following presentation.

[redacted] STRUCTURES BURIED IN SOIL [redacted]

11-7 Definition of Burial Conditions [redacted]

[redacted] Arches and domes that are buried in soil are separated into two primary groups: those that are buried deeply and those that are at shal-

[redacted]

low depths. The criteria defining these groups are shown in Figure 11-25. A fundamental distinction is made between the burial conditions on the basis of the amount of flexure which probably develops as the blast wave envelops the structure. Because it provides a more conservative result, the direction of air-shock propagation is assumed to be perpendicular to the long axis of the structure. When the depth of cover over the crown H_c is less than 6 percent of the span, flexure is important. When the depth is such that the average cover over the entire structure H_{av} is 25 percent of the span, flexure is largely precluded by the resistance mobilized in the soil. Figures 11-26 and 11-27 define the average depths of burial for arches and domes. Flexure becomes progressively less important for intermediate depths, proceeding from shallow to deep buried. An interpolation scheme is suggested in paragraph 11-10 to account for the change in amount of flexure.

[redacted] When the structure is located completely or partially above the ground surface – whether it be an arch, dome, or rectangular structure – with an extensive mound of earth covering it, the classification of deep or shallow burial depends upon the extent of the mound. The general configuration is shown in Figure 11-28. When the slope of the mound, s, as shown in the figure, is 4 or more, the structure is considered to be buried. Whether such a structure is further classified as being buried at a shallow or deep depth depends additionally upon H_c and H_{av}, with the conditions defined in Figure 11-25 controlling the classification for arches and domes. For rectangular structures, the amount of cover over the roof is unimportant in terms of the structural action so long as the slope s is 4 or more.

11-8 Deeply Buried Structures [redacted]

[redacted] Several major phenomena must be accounted for in the evaluation of the vulnerability of a buried structure. Among the more important phenomena are attenuation of air-blast induced pressure with depth, arching of the load from more deformable areas to less deformable ones, virtual mass, and the general structural configuration. Some of these depend mainly on the type of medium and are almost independent of the structure. The situation is reversed for others. Accordingly, the arrangement of the figures in this section ranges from the more general ones, which depend on the medium or on quite general structural characteristics, to quite specific ones which depend on the details of the structural framing and material. The sequence provides a reasonable compromise, allowing the data to be used to analyze a given structure or to define approximate structural dimensions.

- *Pressure Attenuation with Depth.* The data in Figure 11-29 depend on the properties of the soil, which, in turn, define the attenuation of vertical stress with depth. The parameter β is merely the ratio of the side-on overpressure p_{so} to the nearly vertical component of stress p_v at depth H_{av}. Once p_v is known for given conditions, approximate structural dimensions can be evaluated.

The influence of soil type on pressure attenuation is reflected in its effective seismic velocity c_p which is defined in terms of the dynamic constrained modulus of deformation M and the mass density of the medium ρ, as

$$c_p = \sqrt{M/\rho}$$

The constrained modulus of deformation M generally depends on the magnitude of stress considered, and it must be evaluated for each site. For most soils, M is less than the value obtained for elastic conditions; consequently, c_p frequently is less than

[redacted]

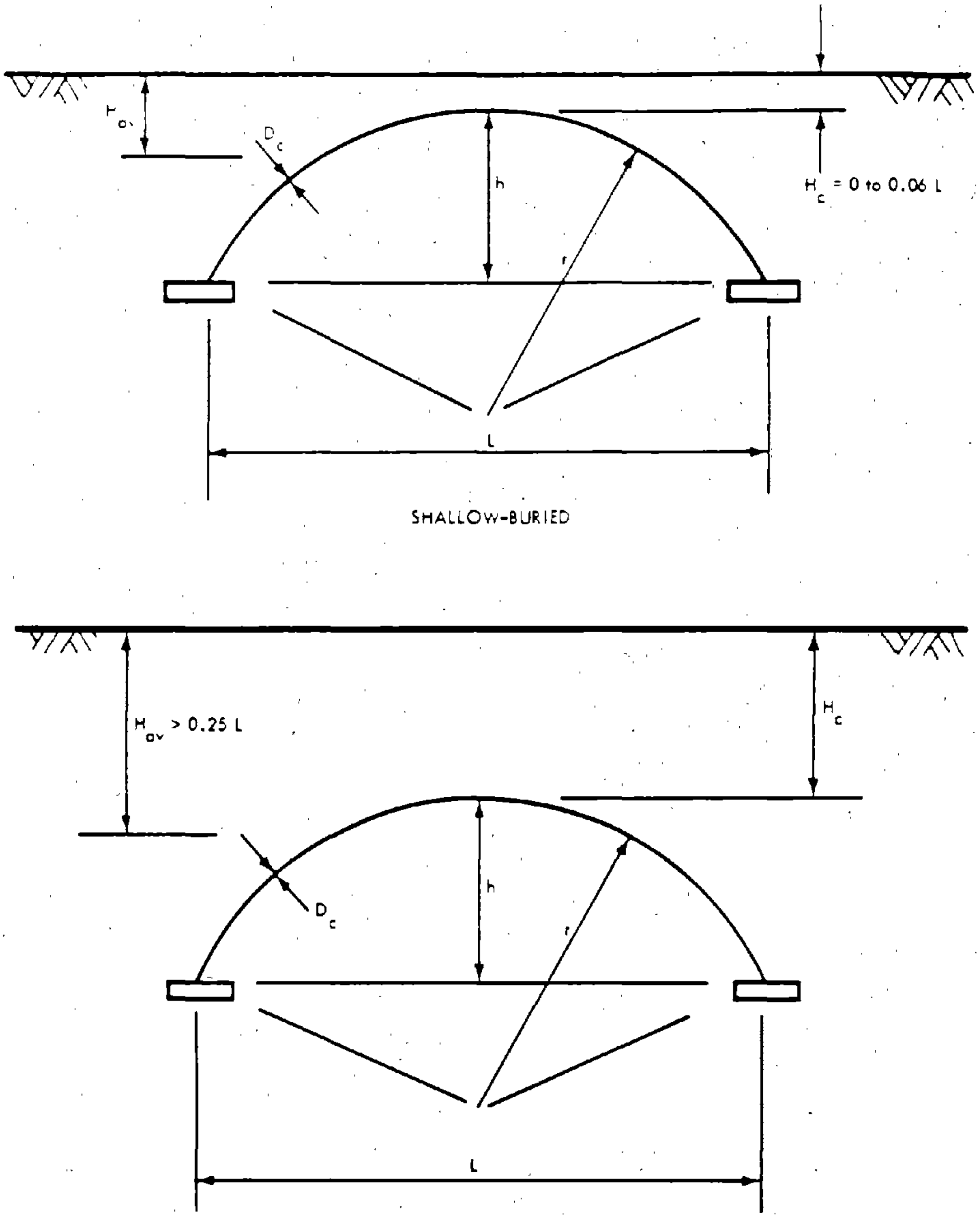


Figure 11-25. [redacted] Depth of Cover Classification for Arches [redacted]

[redacted]

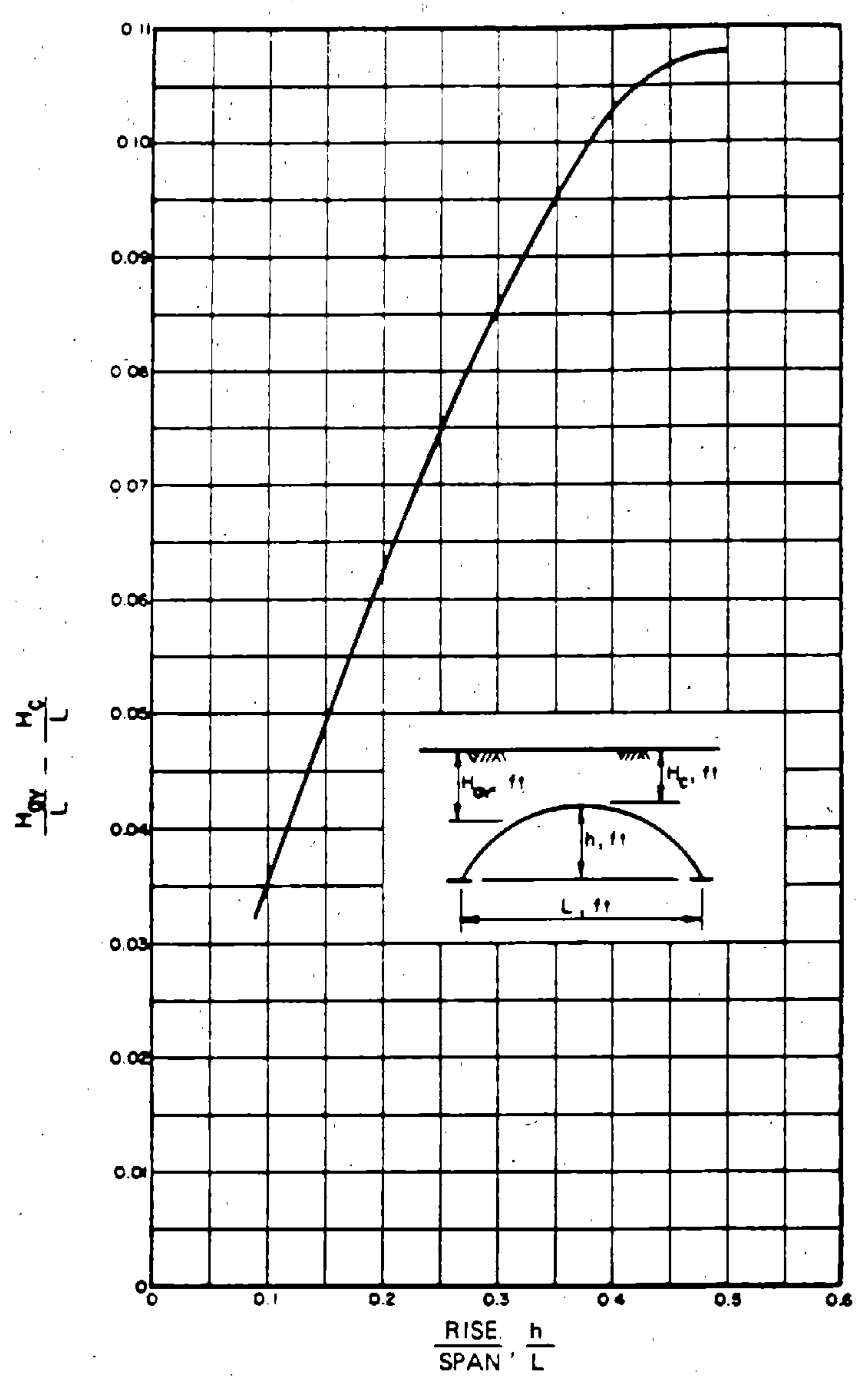


Figure 11-26. [redacted] Buried Arches, Average Depth of Earth Cover [redacted]

[redacted]

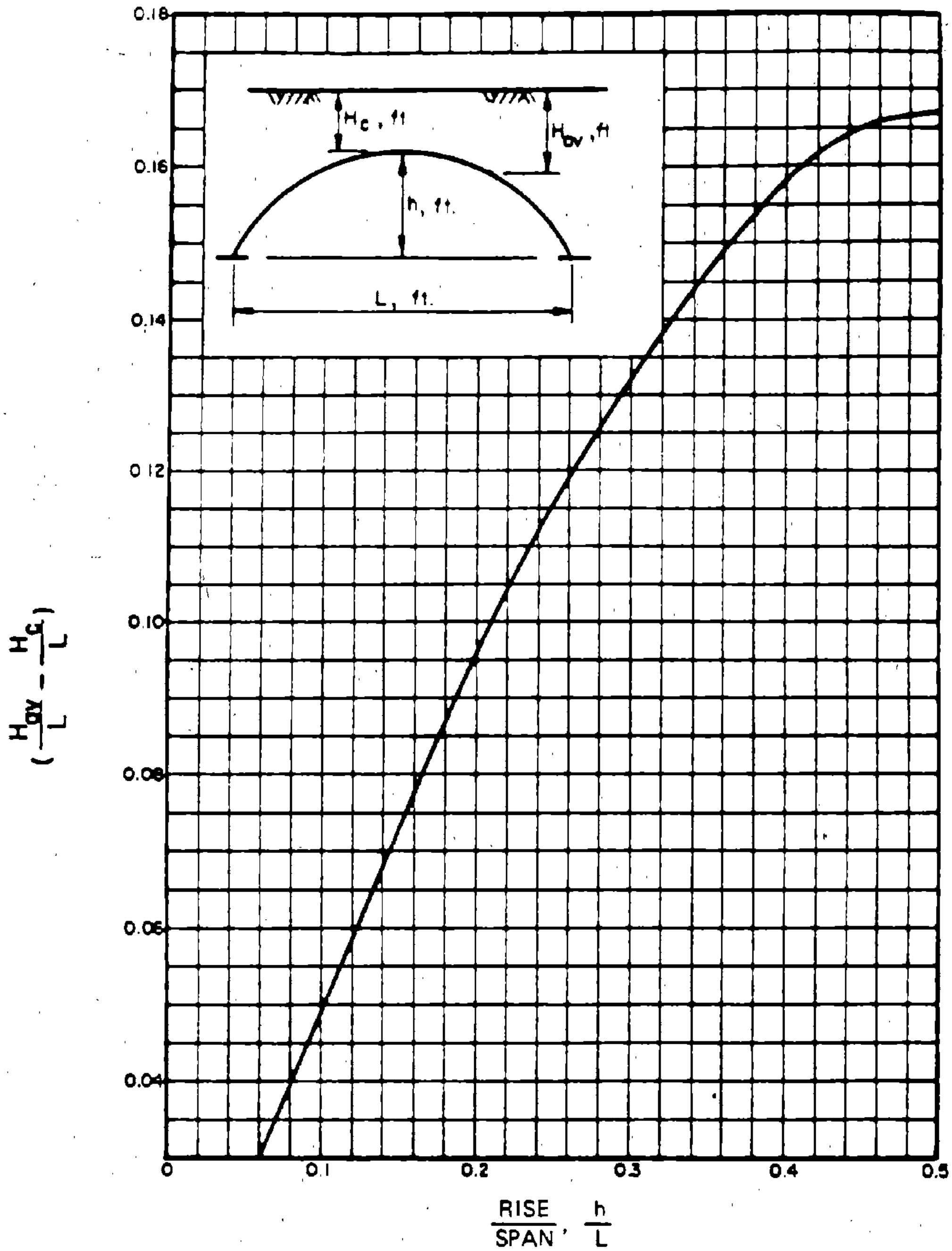


Figure 11-27. [redacted] Average Depth of Earth Cover for Buried Domes [redacted]

[redacted]

[redacted]

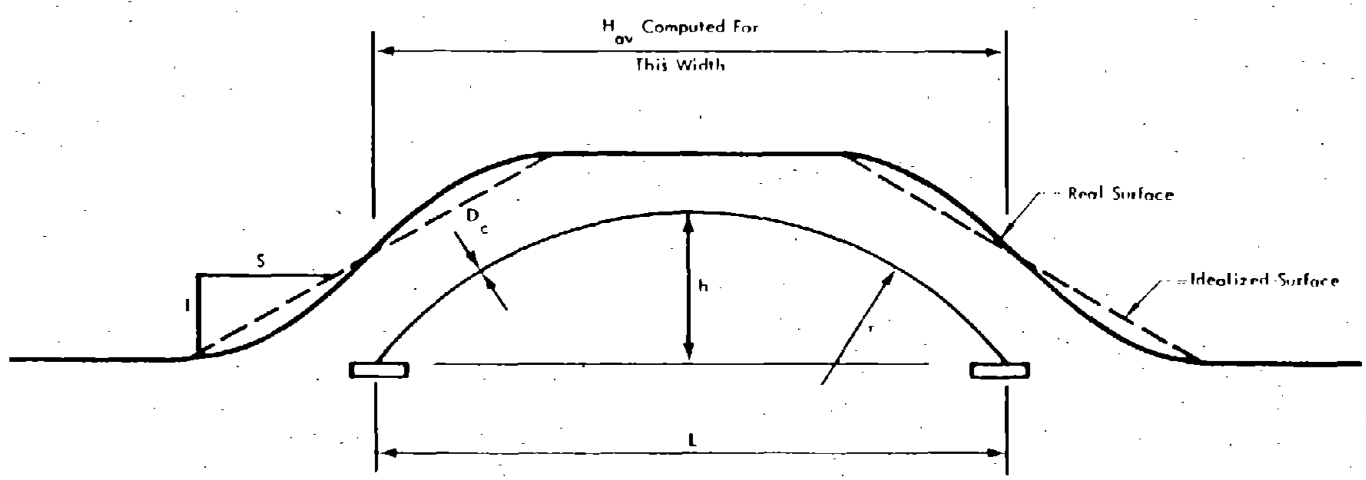


Figure. 11-28. [redacted] Configuration of Mounded Arches [redacted]

[redacted]

the conventional seismic velocity determined by geophysical exploration. This last difference generally is more pronounced for the large stresses encountered in the design of underground structures. Nevertheless, the conventional seismic velocity determined by geophysical methods may be used as a rough approximation of the effective seismic velocity if more complete data are not available.

- *Damage Pressure Level.* Damage pressure level depends upon the general characteristics of the structure as defined by its yield resistance q_y, its natural period of vibration T and its ductility factor μ. The damage pressure level is defined by Figure 11-30 where p_d is the damage pressure level or peak stress applied at the structure-soil interface, and t_d is the effective duration of the loading. For severe damage, a ductility factor of 10 is recommended, with a value of 3 being used for moderate damage.

- *Arching.* Arching depends on the amount of cover over the structure relative to the span L and on the relative deformability of the structure and medium. Because arching factors appear to depend more on degree of damage than on structural configuration or soil type, only two values are shown, one for moderate and the other for severe damage. The arching factor X from Figure 11-31 is multiplied by the yield resistance q_y to define the capacity of a given structure.

- *Effective Duration of Loading.* Figure 11-32 defines the ratio of the effective pulse duration t_d to the effective period of the structure. Both Figures 11-32 and 11-34a and b use the several parameters defined in Table 11-10 and Figure 11-33 to specify input data. For impulsive loads, Figure 11-34a and b provide a direct solution for the side-on overpressure p_{so} for moderate and severe damage, respectively, once the capacity $q_y X$ of the structure is determined.

- *Yield Resistance of Structure.* Figures 11-35 through 11-42 give the yield resistance of particular structural configurations. For example, Figure 11-35, in conjunction with Figures 11-35a and 11-35b defines the yield resistance q_y of deeply buried, horizontal, reinforced-concrete arches or of deeply buried, horizontal, reinforced-concrete cylinders of radius r and average depth of cover H_{av} for the top half of the cylinder. Similarly Figure 11-35 provides an approximate definition of the resistance of a reinforced-concrete capsule consisting of a right circular cylinder capped with hemispherical ends of thickness equal to that of the cylinder. The correction factor in Figure 11-35b is multiplied by the resistance q_y from Figure 11-35 when the properties of the section differ from those in the basic graph.

  For some structures, particularly steel or aluminum arches, domes, or silos at shallow depths, buckling may develop under the dynamic loading. However, these cases are relatively rare, and it is sufficient only to mention the possibility.

- *Vertical Silos and Walls.* The horizontal component of stress p_h at a point below the surface of the soil normally differs from the vertical component as defined by Table 11-6. Consequently, it generally will be necessary to convert the peak applied stress p_d as determined for a given vertical silo or wall to a vertical component of stress p_v by dividing by K_o from Table 11-6 before applying the attenuation factor β from Figure 11-29.

An additional complication arises for vertical silos in that the dead-load stress depends upon the equivalent angle of internal friction of the surrounding soil as indicated by Figure 11-43 in conjunction with Table 11-7. An estimate of the horizontal dead load is determined directly from Figure 11-43 as modified by Table 11-7. However, since Figures 11-35 and 11-36 are constructed with curves parametric in H_{av}, it is necessary to convert p_H from Figure 11-43 to an equivalent average cover by dividing p_H by

$$\frac{120 \text{ lb/ft}^3}{144} = 0.8 \text{ psi/ft},$$

the assumed unit weight of soil. With this equivalent H_{av} Figure 11-35 or 11-36 can be used directly to determine the capacity of a vertical silo.

Near the surface, at depths above those that are comparable to the structural diameter, it is necessary to provide additional strength in a vertical silo to resist the inevitable nonuniform load that develops as the air shock envelops the structure. Normally, the peak value of this nonuniform load is taken equal to about one-half of the peak side-on overpressure at the ground surface, diminishing linearly to zero at a depth equal to the silo diameter, and lasting for the time of transit across the structure. This nonuniform load is assumed to have the form of two complete sine waves around the circumference. Analysis for this nonuniform loading can be accomplished by methods discussed below for structures buried at shallow depths.

The structure also must have sufficient strength in the vertical direction to withstand a sizable skin friction force. Generally this force does not control the dimensions of the wall, but special reinforcement may be required to resist the forces.

- *Structural Dimensions ("Design").* It is possible to determine approximate dimensions of a structure directly by following these general steps in conjunction with the figures and tables described previously:
  (1) From the given p_{so}, H_{av}, and W and an estimate of c_p for a given site, find β from Figure 11-29.
  (2) Calculate $p_v = p_{so}/\beta$.
  (3) Find t_d/T from Figure 11-32 in conjunction with Table 11-10 and Figure 11-33.
  (4) Find $p_d/q_y X$ from Figure 11-30 or $q_y X$ from Figures 11-34a or b, depending on the value of t_d/T.
  (5) Find X from Figure 11-31.
  (6) Calculate q_y.
  (7) Find the approximate structural dimensions from the appropriate figure (11-35 through 11-42) modified by Table 11-6 and/or by Figure 11-43 if appropriate.

- *Vulnerability Evaluation ("Analysis").* "Analysis" of a structure is not quite so straightforward, because the side-on overpressure at the surface generally is needed in intermediate steps. Since this overpressure frequently is the answer desired, a trial and error process is required; an overpressure is assumed and the steps below are followed to find a new overpressure. The steps are:
  (1) From given structural dimensions, find q_y from the appropriate figure (11-35 through 11-42) modified by Table 11-6 and/or by Figure 11-43 if appropriate.
  (2) Find X from Figure 11-31.

[redacted]

(3) Assume p_{so} and find t_d/T from Figure 11-32 in conjunction with Table 11-10 and Figure 11-33.

(4) Find $p_d/q_y X$ from Figure 11-30 or p_{so} from Figures 11-34a or b, depending on the value of t_d/T. If either Figure 11-34a or 11-34b is used, compare p_{so} with that assumed in step (3); repeat step (3) as required.

(5) Compute p_d if Figure 11-30 is applicable.

(6) Find β from Figure 11-29 if Figure 11-30 is used in step (4).

(7) Calculate $p_{so} = \beta p_d$ and compare with that assumed in step (3). Repeat steps (3) through (7) as required.

- *Illustrative Examples.* These general procedures for "design" and "analysis" are illustrated in Problems 11-3 and 11-4, respectively.

11-9 Structures Buried at Shallow Depths [redacted]

[redacted] Evaluation of the vulnerability of structures at shallow depths is complicated by the requirement for superposition of flexure and direct or hoop compression. In the case of deeply buried structures only direct stress was considered as effective for shell structures, such as arches; only flexure was considered effective for rectangular elements.

[redacted] Because the figures and tables described earlier are restricted either to direct stress or to flexure, they may be used to define basic parameters required in the analysis of structures that are buried at shallow depths. For example, from Figure 11-35, the resistance to pure thrust is defined directly for a reinforced concrete arch of thickness D_c; Figure 11-38 defines directly the resistance to pure flexure of a one-way slab of span b_s and thickness D_c. Thus, if the span b_s is appropriately defined, Figures 11-35 and 11-38 define key input parameters for entering an interaction diagram such as Figure 11-44. In this diagram P_u, the total capacity under *pure* thrust, is proportional to q_y as determined from Figure 11-35, and M_p, the total capacity under *pure* bending, is proportional to q_y as determined from Figure 11-38. If the corresponding numerators P and M, respectively, are defined properly, the necessary conditions are available to use Figure 11-44.

[redacted] Actual solution for an arch buried at shallow depth involves a trial and error process. Whether "analysis" or "design" is involved, the first step requires knowledge of the structural dimensions. From the known properties of the structure, the resistance to thrust alone is found from Figure 11-35 in conjunction with Figures 11-35a and 11-35b. The resistance to flexure alone is found from Figure 11-38, assuming a span b_s equal to one-half the developed length of the arch.

[redacted] The loading which produces the thrust is the vertical component of the stress in the medium p_v. Since by definition of shallow burial the structure is just below the surface, attenuation with depth can be ignored, and p_v can be taken equal to a given or assumed value of p_{so}. The appropriate effective duration is found from Figures 11-32, Table 11-10, and Figure 11-33. In turn, the required q_y for this loading alone is found from Figure 11-30 or 11-34a or b. The ratio of this q_y to the one found above from Figure 11-35 is equivalent to the ratio P/P_u in Figure 11-44.

[redacted] The loading which is assumed to produce flexure may be taken as one-half of the given or assumed value of peak side-on overpressure. This loading is effective only during the time of transit across the structure; thus, its duration is L/U where U is the air-shock velocity (see paragraph 2-15, Chapter 2). If the quantity

$$\psi\left(\frac{10\ \text{ft}}{L}\right)\left(\frac{U}{1{,}000\ \text{fps}}\right)$$

[redacted] is used instead of the values of γ obtained from Table 11-10, this value of γ may be used in Figure 11-32 to find the appropriate t_d/T. With this value, the required q_v' for this loading alone is found from Figure 11-30 or Figure 11-34a or b. The ratio of this q_v to the one found above from Figure 11-38 is equivalent to the ratio M/M_p in Figure 11-44.

[redacted] With the values of P/P_u and M/M_p now known, Figure 11-44 is entered to determine if the assumed p_{so} is valid for a problem in "design." If the assumed value is not adequate in either situation, the process is repeated making new assumptions as indicated by the solution.

[redacted] The case just discussed is the more complicated of those which might be encountered. The other case of primary interest concerns the dome at shallow depth. For this type of structure, the flexure, which has a duration and magnitude as noted above for the arch, is carried by membrane action in a manner similar to the thrust produced by the uniform load. Thus, the interaction diagram of Figure 11-44 is not applicable. The thrust produced by the uniform component of load is carried in the same fashion as that for a deeply buried dome. The thrust caused by flexure is carried by membrane action with the induced stress being approximately twice that produced by the uniform radial pressure of the same peak intensity. Thus, Figure 11-37, for example, can be used to define this "flexural thrust" by multipling the result from this figure (either q_v or D_c/r since they are essentially proportional) by 2. The thickness of the shallow-buried dome must be sufficient to withstand both induced thrusts simultaneously.

[redacted] The procedures for analyzing the vulnerability of structures buried at shallow depths are illustrated in Problems 11-5 and 11-6.

11-10 Intermediate Depths of Burial [redacted]

[redacted] When a structure is buried at depths intermediate between shallow and deep, it is sufficient to find its capacity at either extreme and to interpolate linearly between the overpressures at either extreme using the average depth of cover H_{av} as the interpolation parameter.

[redacted] LINED AND UNLINED OPENINGS IN ROCK [redacted]

[redacted] Several analytic solutions are available that give stresses around a cylindrical opening in an elastic medium when it is engulfed by a stress wave. In addition, experimental data from two major field experiments in rock are available. Correlation of analytic and experimental results is difficult, primarily because of the simplifying assumptions in the analytic solution that the medium is linearly elastic, homogeneous, and isotropic. Rock, in place, does not meet these requirements. In addition, it is very difficult to assign a specific strength to the natural rock. In fact, the strength probably varies with distance from the opening and with the size of the opening relative to the average joint spacing around the opening. For these reasons, the vulnerability evaluations described in succeeding paragraphs are based primarily upon the experimental data, with the analytic work, tempered with judgment, being used to extrapolate the basic data to other ranges, rock types, and structural sizes and shapes. Almost all the basic data were obtained for openings that were circular cylinders in a granite medium. Actual structures in rock would be placed in openings that are as near circular as possible, because this shape provides the smallest theoretical stress concentration. Therefore, only openings that are essentially circular will be considered.

[redacted] The limitation of the material presented should be kept in mind. The basic data were obtained for several different structural configurations, but there were only two weapon yields, a limited number of ranges from the burst point, and only one site. It is known that the particular site conditions will affect the [redacted] strength of an opening even in the same type of medium, and this has been taken into account by considering the site to be average in jointing, faulting, and rock strength. As in the preceding sections, the probability of the stated damage is 50 percent.

[redacted] One of the greatest uncertainties in using the experimental data arises from the fact that the energy source in the specific experiments was deeply buried, and the relative release of energy by deeply buried bursts and surface bursts is not understood completely. The recommendations in this section apply to a contact surface burst, and the energy associated with the surface burst is taken to be 0.05 of the total energy in the underground burst.

[redacted] High stresses in some regions around the opening may cause rock crushing that would result in a redistribution of stress in the vicinity, but the damage may not be sufficient to impair the use of the opening; this may be particularly true if this possibility is recognized and provision is made to accommodate the extra rock volume resulting from the crushing. Dead-load stresses from the rock overburden are normally considered in the stress-distribution evaluation around an opening in rock. Tectonic forces occur in most rock masses, but their magnitude and direction is so uncertain that the inclusion of these effects is impractical.

[redacted] In view of the many uncertainties, it is impossible to assign a single range to a particular damage level for a given structure in a given medium. Therefore, lower and upper limits have been set in each case, which, it is believed, will bound the condition for any particular problem. By examining the conditions at a given site in those cases where sufficient information is available, it should be possible to determine whether the structural vulnerability should lie toward the upper or lower bound. Also, for average conditions, the end of the range corresponding to the largest distances from the detonation can generally be considered to apply to multiple attack conditions. For attacks that are directly overhead, the required range should be added to the depth of crater obtained in the first attack to give the necessary range for survival from multiple attacks. This crater depth may be obtained from Section II, Chapter 2.

[redacted] The tables and the figure provided to estimate damage to structures buried in rock should not be extrapolated below approximately 20 kt since the mechanism of damage for smaller yield weapons involves such small vulnerability radii that the mechanism of damage is changed considerably from that assumed, and a direct hit is required to inflict any significant damage.

11-11 Definition of Damage to Openings in Rock [redacted]

[redacted] Near the detonation on a rock surface the material will be crushed and rearranged rather violently. However, some radius will be reached past which there will be little visible damage to the rock even though it has been subjected to rather high stresses, since it is completely contained and the strength is thus increased. In the zone where there is crushing of rock, it is probable that no practical structure can be built to prevent the collapse of an opening. However, in the region outside the zone of crushing it is possible to strengthen an opening with a structural lining. The damage to the tunnel or size of weapon required to inflict damage then depends upon the strength of the lining and the depth of the tunnel, since the stress wave intensity diminishes with depth. The choice of depth then becomes an economic one in which the added cost of greater depth must be balanced against the reduced cost of the requirement for weaker and consequently less expensive structural liners.

[redacted] Damage to unlined openings directly outside the zone of crushing would consist of complete closure of the opening by rock that

[redacted]

has failed. As the distance from ground zero is increased, the damage resulting from this mechanism becomes less until the stress level is reduced to the dynamic unconfined compressive strength of the rock, at which point there is no damage from rock that has failed.

[redacted] At ranges where an opening remains, high speed photography in field experiments confirms the presence of rock fragments which move with sufficient velocity to render the equipment and personnel inoperative in an unlined tunnel. Although the wavelengths associated with the pulses imply sizes of spall that are quite large, natural jointing, irregular shapes of opening, and possible destressing could result in tensile spalling of the tunnel surface. When a compression wave intersects a free surface, a tensile wave is generated which travels back into the medium. High frequency undulations may be superimposed on the smooth wave normally assumed. Whenever the tension exceeds the compression locally by an amount equal to the ultimate tensile strength of the rock, a spall can form. A spall having a depth approximately greater than the radius of the opening will not be able to dislodge and fly into the tunnel.

[redacted] Because of jointing and faults in the rock there is a tendency for loose rocks to be dislodged and to drop into the tunnel. This type of damage will be present so long as a sufficiently large opening is created to allow a previously loosened rock to drop. The amount and size of rock that drops into the opening will depend on the amount of disturbance of the rock and would therefore be maximum near the detonation and smaller with increased distance. This damage would extend out to a greater distance than either of the previous two types of damage discussed. Rocks which result from this mechanism of failure can be distinguished from the spalls discussed above because they drop by gravity while the spalls have an initial velocity.

11-12 Types of Tunnel Linings [redacted]

[redacted] Two different general approaches have been taken in the construction of tunnel linings. These techniques are described separately as follows:

- *Integral Linings.* A lining may be constructed in contact with the rock face and with sufficient strength to confine the rock in the radial direction and thereby strengthen it in the hoop direction, which is the direction of highest stress. The type of lining designed intentionally to confine the surrounding rock lends protection from all three types of damage (closure, spalling, and rock drops). The lining may be constructed of reinforced concrete cast against the rock, steel linings with concrete placed behind them, or a layered combination of steel and concrete.

  Rock bolts around the opening designed to provide a radial force on the opening face may act in the same general way as an integral lining but the radial stress developed by rock bolts is generally much less than that developed by an integral lining.

- *Linings with Packing.* A lining may be built with a cavity between the lining and the rock, which is filled with a highly deformable material. This procedure has the advantage that local crushing and partial failure of the rock can occur around the opening with the resulting volume increase of rock being absorbed by deformation of the packing, which is chosen to have elasto-plastic stress-strain characteristics. The packing also tends to smooth the pressure on the outside of the lining and to limit its magnitude until the elasto-plastic capacity of the material is exceeded. If a fairly general crushing failure around the opening is allowed to develop, it has been [redacted] found that a lining surrounded with a rather thick packing can be kept open at much closer radii relative to the point of burst than can a practical integral liner. The radial confinement of the rock by the packing material is rather insignificant in this case, and it is not considered in the design. The primary design considerations are the amount of crushing that will occur in the vicinity of the opening, the increase in volume of the rock that will occur upon crushing, and the amount of packing that must be provided to absorb this volume change.

If a tunnel is sufficiently far below the surface that the stress level in a wave that can reach it is very small, or the protection level required is not great, it may only be necessary to protect against damage by dropping of rocks. This may be done by lining the opening with woven wire fence and rock bolting this fence to the wall. Such a system can be made fairly strong if a sufficient number of rock bolts are used. Another possibility is to use steel sets and timber lagging similar to those used in conventional mining practice. The basis of design for this type of protection consists mainly of experience and the competency of the rock mass.

11-13 Procedures for Tunnel Vulnerability Evaluation [redacted]

[redacted] Table 11-8 describes tunnels with several types of linings in granite. Estimated ranges beyond which survival will occur after a 1 Mt surface burst are also shown. The linings considered include a special composite lining for survival at very close distances, a composite lining similar to that for which experience is available from Operations HARD HAT and PILE DRIVER, a modified composite lining to permit somewhat smaller openings to be used, a special integral lining, and an ordinary integral lining generally of concrete placed directly against the rock, a section with rock bolts and mesh only, and, finally, an unlined section with a thin gunite coating over the interior of the tunnel. The slant ranges for survival of these various linings are given for granite with a seismic velocity of about 18,000 ft/sec.

[redacted] For depths less than the slant ranges shown in Table 11-8, a horizontal distance is implied that will give the proper slant range for survival. It is likely that the survivability ranges of structures very near the surface may be less than those that correspond to the slant ranges shown in Table 11-8 as a result of the direct transmitted motion alone. However, because of a lack of data, and other effects which must be taken into account, it is recommended that the same distances be used for horizontal ranges for relatively shallow tunnels. Table 11-8 can be used to estimate the ranges for vulnerability of various types of liners by comparing a design with the typical designs given in the table and by interpolating between the designs.

[redacted] It is believed to be possible for a tunnel to survive at a range as small as 600 to 700 feet from a 1 Mt explosion with a very expensive type of lining construction for relatively small diameters, whereas the range might be as large as 1,600 to 1,800 feet if the tunnel is completely unlined, with the stipulation that survivability is assured only if no local rock failure occurs. Hence, estimates of tunnel vulnerability can be made directly for various depths of cover and corresponding ranges provided that the system vulnerability can be properly placed relative to those given in Table 11-8 and that the local rock conditions can be assessed relative to average rock conditions as used in the table. This can be done as discussed in the following paragraphs.

[redacted] The free-field stress that can be tolerated in the area of a protective structure in rock depends upon: the average closure of the opening that can be tolerated by the structure contained within the rock opening; the "effective" stiffness, or Young's modulus, of the jointed medium; and the strength parameters of the jointed rock mass. The free field stress also will be a function of the range, yield, and medium in which the detonation takes place. All of the above factors will determine the range from a given explosion at which various types of structural configurations can survive in a rock mass. Each of the factors will be discussed separately, and a procedure to arrive at a given design will be suggested. It is beyond the scope of this manual to provide complete quantitative data for the design of structural configuration that will survive in a rock mass. Therefore, the following discussion will merely outline the procedures. For more detail, the users of the manual should consult the references that are cited. Estimates of the vulnerability of such structures may be obtained from Table 11-8, as described in the preceding paragraph, or from Figure 11-45 as described below.

- *Tolerable closure for a given rock opening.* The average closure of an opening is expressed by the diametrical strain across the opening $\Delta r/r$, where r is the radius of the opening. The diametrical strain is equal to the circumferential strain, ϵ_θ, at the edge of the opening. For back packed structures, the tolerable change in radius Δr is a function of the thickness of back packing and the locking strain in the back packing. In the case of integral structural linings the permissible deformations are smaller and are a function of the type of lining. For example, the limiting circumferential strain, ϵ_θ, for a concrete liner may be 0.003 and the limiting strain, ϵ_θ, for a stiffened steel integral lining might be 0.01.
- *"Effective Stiffness."* The "effective" stiffness of a jointed mass is a function of both the "intact" rock properties and the spacing and character of the discontinuities. A method for selecting the deformation modulus of a rock mass has been given by Deere, Hendron, Patton and Cording (see bibliography). By this method the rock quality of the rock mass must first be assessed quantitatively in terms of the Rock Quality Designation (RQD) or the Velocity Ratio (see Deere, et al., bibliography). After the rock quality has been determined, the reduction factor, which is the ratio of the deformation modulus, E_r, to the dynamic value of Young's modulus, E_{seis} (which is calculated from P wave velocities measured in seismic surveys), is determined as follows:

$$\frac{E_r}{E_{seis}} = 0.14, \text{ for } 0 < RQD < 0.69,$$

$$\frac{E_r}{E_{seis}} = 2.77\, RQD - 1.77, \text{ for } 0.69 \leq RQD \leq 1.$$

If the RQD of the rock mass and the Young's modulus from intact rock cores, E_{core}, are known, the reduction factor E_r/E_{core} may be determined approximately from

$$\frac{E_r}{E_{core}} = 2.06\, RQD - 1.23.$$

Once either reduction factor has been determined, E_r may be determined from the product of the RQD and either E_{seis} or E_{core}.

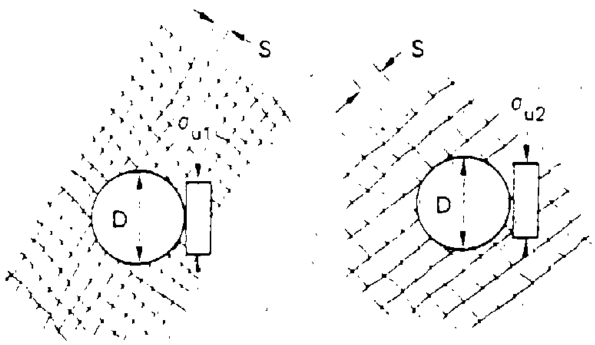
- *Strength of a jointed rock mass.* The strength of a jointed mass surrounding a tunnel can be given by

$$\sigma_1 = \sigma_u + N_\varphi \sigma_3,$$

where σ_1 and σ_3 are the major and minor principal stresses at failure, σ_u is the "effective" unconfined strength of a jointed rock mass, and

$$N_\varphi = \frac{1 + \sin\varphi}{1 - \sin\varphi}.$$

The angle φ is defined as the angle of shearing resistance along the planar discontinuities in the rock mass. It should be noted that the value of φ along the discontinuities is significantly lower than the value of φ that would be derived from triaxial tests on intact samples of rock. The value of the unconfined compressive strength of the rock mass, σ_u, is a function of the ratio of tunnel diameter to joint spacing as illustrated below, where



(1) (2)

$$N_{\varphi 1} \approx N_{\varphi 2} \approx \varphi_{\text{joints}},$$

and

$$\sigma_{u1} < \sigma_{u2} = f(D/S).$$

The larger the ratio of tunnel diameter to joint spacing, D/S, the smaller the value of σ_u appropriate for design. If D/S is very small, the value of σ_u can approach the unconfined strength, q_u, of intact samples of the rock surrounding the tunnel. Similarly, if D/S is very large, σ_u approaches zero and the shear strength of the rock mass approaches the shear strength along the joints. The relationship between σ_u/q_u and D/S is

$$\frac{\sigma_u}{q_u} = (-0.08)\,\frac{D}{S} + 1.$$

Thus, if q_u and the ratio D/S are known, a value may be selected for σ_u for design.

- *Step-by-step method of determining vulnerability.* (1) The following expression can be derived from the relations between radial strain and velocity, acceleration, or displacement given in paragraph 2-64, Chapter 2, and the separate relations given in the same paragraph for radial acceleration, velocity, and displacement:

$$\epsilon_r = 0.0015\ W(\mathrm{Mt})^{5/6} \left(\frac{1{,}000}{R}\right)^{5/2} \left(\frac{165}{\gamma}\right)^{5/6} \left(\frac{18{,}000}{c_p}\right)^{5/3},$$

where, as in Chapter 2, ϵ_r is the free-field radial strain, R is the radial distance in feet, γ is the unit weight of the medium in pounds per cubic foot, c_p is the seismic velocity of the medium.

(2) Calculate the free-field stress, σ_o,

$$\sigma_o = \left(\epsilon_r\right)\left(E_r\right)\left(\frac{(1-\nu)}{(1+\nu)(1-2\nu)}\right).$$

[redacted]

where E_r is the "effective" Young's modulus of the jointed medium, and ν is Poissons' Ratio. Normally ν may be taken to be 0.3, so

$$\sigma_u' \approx 1.35\ \epsilon_r E_r .$$

(3) Use the value of the free-field stress, σ_o, and the properties of the rock mass, E_r, ν, σ_u, and N_ϕ to perform an elastic-plastic analysis (see Hendron and Aiger (1971) or Newmark (1969) in bibliography f to determine the circumferential strain ϵ_θ in the rock at the boundary of the rock-tunnel surface. Note that $\epsilon_\theta = \Delta r/r$, where r is the radius to the rock surface.

(4) Compare this strain to the strain which can be tolerated by the structural system inside the rock opening.

[redacted] The available information on the maximum ranges at which the types of damage discussed in paragraph 11-11 occur in granite and shale is shown in Figure 11-45. Also shown on this figure are the ranges in which the various types of linings discussed in this section could be used and could be constructed to survive at the scaled ranges shown. At smaller scaled ranges than those shown, it is believed that the installation would no longer be able to function. This figure shows the wide variation in the competency of natural deposits of rock, though one would expect a protective system to be placed in a rock with properties which lie toward the upper part of the range. Also, the effect of rock strength on the scaled range beyond which survival can occur can be seen from this figure. The steepness of the curve is also of particular note. Granite and shale were chosen for this figure as representative of a strong and a relatively weak rock, respectively. These procedures are illustrated in Problem 8-7.

11-14 Vulnerability Evaluation of Surface Silos in Rock [redacted]

[redacted] The comments made in the preceding paragraphs concerning deeply buried tunnels in rock are generally applicable to surface silos in rock. [redacted] the three types of damage discussed in paragraph 11-11 still occur, and the linings discussed in paragraph 11-12 act in the same way. [redacted] Therefore, it is recommended that the same vulnerability distances given in Table 11-8 be used for horizontal ranges for vertical silos or other openings with linings that are similar to those in Table 11-8. A summary of the effects on surface silos designed for various kinds of lining is shown in Table 11-9. In addition, Figure 11-45 can be used to obtain the approximate scaled ranges for which various types of damage will occur.

DNA (b)(1)

DNA (b)(1)

[redacted]

Table 11-6. [redacted] Ratio of Horizontal to Vertical Soil Pressures, K_o [redacted]

Condition of Soil	K_o, For Stresses up to 1,000 psi		
	Dynamic	Static	
	Undrained	Undrained	Drained
Cohesionless soils, damp or dry	1/4	1/3-dense 1/2-loose	1/3-dense 1/2-loose
Unsaturated cohesive soils of very stiff to hard consistency	1/3	1/2	1/2
Unsaturated cohesive soils of medium to stiff consistency	1/2	1/2	1/2
Unsaturated cohesive soils of soft consistency	3/4	1/2 to 3/4	1/2 to 3/4
Saturated soils of very soft to hard consistency and cohesionless soils	1	1	1/2-stiff 3/4-soft
Saturated soils of hard consistency, q_u = 4 tons/sq ft to 20 tons/sq ft*	3/4 to 1	1	1/2
Saturated soils of very hard consistency, q_u > 20 tons/sq ft	3/4	1	1/2
Rock	Obtain from tests on rock cores and correlate with seismic data.		

[redacted] q_u = ultimate bearing capacity.

[redacted]

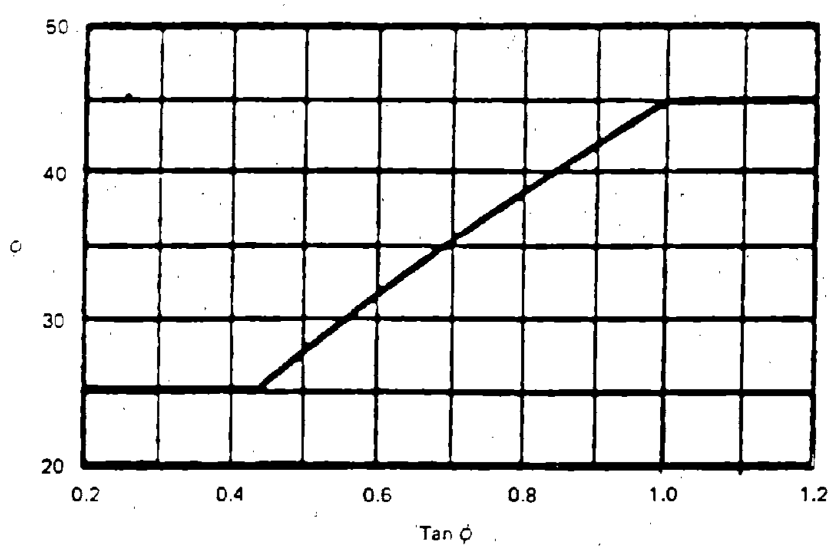
[redacted]

Table 11-7. [redacted] Equivalent Friction Angles for Nongranular Cohesive Soils [redacted]

Consistency of Soil	Average Unconfined Compressive Strength (tons per sq ft)	Equivalent* z Tan ϕ (ft)
Very soft	0.15	1.2
Soft	0.40	3.3
Medium	0.75	6.2
Stiff	1.50	12.5
Very stiff	3.00	25.0

* [redacted] Determine ϕ from these values to enter Figure 11-43. If value of ϕ thus determined is less than 25°, use 25°; if more than 45°, use 45°.

After "z Tan ϕ" from the tabulation above has been divided by the depth from the ground surface z to obtain "Tan ϕ," the value of the equivalent friction angle ϕ may be determined from the following curve:



[redacted]

[redacted]

Table 11-8. [redacted] Typical Tunnel Design Concepts [redacted]

Deleted

[redacted]

[redacted]

Table 11-9. [redacted] Limits of Vulnerability in Terms of Overpressure for Surface Silos — 1 Mt [redacted]

Deleted

[redacted]

[redacted]

Problem 11-3 Calculation of the Thickness for a Deeply Buried Reinforced Concrete Dome

[redacted] Figures 11-29 through 11-43 and Tables 11-6, 11-7, and 11-10 provide the information necessary to determine approximate dimensions of a deeply buried structure that will withstand a prescribed level of damage from a specified threat.

[redacted] *Example* [redacted]

Given: A deeply buried hemispherical dome, r = 30 feet, with a required span L = 60 feet, in soil having a seismic velocity c_p = 1.000 fps. The average depth of cover, H_{av}, is 20 feet (if H_{av} is not provided, an estimate may be obtained from Figure 11-28).

Find: The required thickness of the structure that will allow it to sustain no more than moderate damage from a 1 Mt burst at a distance where the peak side-on overpressure, p_{so}, is 1.000 psi at the surface of the ground.

Solution: The solution follows the step-by-step procedure given in paragraph 11-8, Structural Dimensions ("Design").

Step (1):

$$\alpha = \left(\frac{H_{av}\ (\text{ft})}{370\ \text{ft}}\right)\left(\frac{1}{W(\text{Mt})}\right)^{1/3}\left(\frac{1.000\ \text{fps}}{c_p\ (\text{fps})}\right)$$

$$= \left(\frac{20}{370}\right)\left(\frac{1}{1}\right)\left(\frac{1.000}{1.000}\right) = 0.054$$

From Figure 11-29, for α = 0.05 and p_{so} = 1.000, β = 1.1, and it does not depend on p_{so} very markedly.

Step (2):

$$p_v = \frac{p_{so}}{\beta} = \frac{1.000}{1.1} = 900\ \text{psi}.$$

Step (3):

From Table 11-10 ψ for arches and domes is

$$\psi = \left(\frac{10\ \text{ft}}{r}\right)\left(\frac{c_m}{1.000\ \text{fps}}\right)$$

$$= \left(\frac{10}{30}\right)\left(\frac{2.700}{1.000}\right) = 0.9$$

where c_m = 2.700 fps for a concrete dome (Table 11-10).

$$\gamma = \psi\ (W(\text{Mt}))^{1/3}\left(\frac{100\ \text{psi}}{p_{so}}\right)^{7/8}$$

$$= 0.9\left(\frac{1}{1}\right)\left(\frac{100}{1.000}\right)^{7/8} = 0.12.$$

γ' is determined to enter Figure 11-32:

$$\gamma' = \left(\frac{H_{av}}{370\ \text{ft}}\right)\left(\frac{1}{W(\text{Mt})}\right)^{1/3}\left(\frac{1.000\ \text{fps}}{c_p}\right)\left(\frac{p_{so}}{100\ \text{psi}}\right)^{1/2}$$

$$= \left(\frac{20}{370}\right)\left(\frac{1}{1}\right)\left(\frac{1.000}{1.000}\right)\left(\frac{1.000}{100}\right)^{1/2} = 0.17.$$

From Figure 11-32 $t_d/T > 3$; therefore $p_d/q_v X$ = 1.

Step (4):

$$\frac{p_d}{q_v X} = 1 \text{ from the preceding step.}$$

Step (5):

$$\frac{H_{av}}{L} = \frac{20}{60} = 0.33$$

From Figure 11-31 X = 1.0.

Step (6):

$$\frac{p_d}{q_v X} = \frac{p_d}{q_v(1)} = \frac{p_d}{q_v} = 1\ \text{(Moderate Damage)}.$$

Therefore

[redacted]

[redacted]

[redacted]

$$q_y = p_d = 900 \text{ psi.}$$

Step (7).

From Figure 11-37, $D_c/r = 0.097$.

Answer: The required thickness of the hemispherical dome that will allow it to withstand the prescribed environment is

$$D_c = 0.097r = 0.097 \times 30 = 2.9 \text{ feet.}$$

This answer applies to average properties of the materials, i.e., the ratio of total steel in one direction to the effective concrete area, $p_t = 0.005$, the static compressive strength, $f'_c = 4{,}000$ psi, and the static yield strength of the steel, $f_y = 40{,}000$ psi. The resistance, f_y, should be corrected by factors obtained from Figure 11-37b before entering Figure 11-37 if the material has properties that differ from the average.

Related Material: See paragraph 11-8. See also Sections I and III of Chapter 2.

[redacted]

Problem 11-4 Calculation of the Vulnerability of the Roof of a Deeply Buried, Reinforced Concrete Rectangular Structure

[redacted] Figures 11-29 through 11-43 and Tables 11-6, 11-7, and 11-10 provide the information necessary to determine the vulnerability of a deeply buried structure. As pointed out in paragraph 11-8, these analyses must be performed by a trial and error process as demonstrated in the following examp[redacted]

[redacted] *Example* [redacted]

Given: A rectangular, reinforced concrete structure that is buried under 5 feet of earth cover. The roof is a one-way slab, and the thickness of the supporting walls is such that continuity is negligible, even though much of the reinforcement is carried from the roof to the walls. Other pertinent data concerning the structure and the medium are:

Clear span, b_s = 20 feet
Roof thickness, D_c = 4 feet
Steel reinforcement ratio, p_f = 0.012 (i.e., 1.2 percent)
Yield strength of reinforcement, f_y = 60,000 psi
Effective seismic velocity of the medium, c_p = 2,500 ft/sec.

Find: The side-on overpressure at the surface resulting from the explosion of a 50 kt weapon that will cause severe damage to the roof of the structure.

Solution: The solution follows the step-by-step procedure given in paragraph 11-8 for Vulnerability Evaluation ("Analysis").

Step (1):

$$\frac{D_c}{b_s} = \frac{4}{20} = 0.2.$$

$$p_f\, f_y = 0.012 \times 60{,}000 = 720 \text{ psi.}$$

From Figure 11-38,

$$q_y = 440 \text{ psi}$$

FOR RESTRAINED EDGES: since this structure has simple support, this value must be reduced by one-half (see note in insert of Figure 11-38).

Therefore,

$$q_y = 0.5 \times 440 = 220 \text{ psi.}$$

Step (2):

$$\frac{H_{av}}{L} = \frac{5}{20} = 0.25$$

From Figure 11-31,

$$X = 1.0$$

Step (3):

Assume p_{so} = 200 psi.

The following relationships are obtained from Table 11-10:

$$\frac{1}{\psi} = 100\frac{L^2}{K}\sqrt{1 + \Gamma}$$

for one-way slabs.

$$H_{av} < L; \text{ therefore,}$$

$$\Gamma = 0.8\,\frac{H_{av}}{D_c} = 0.8\left(\frac{5}{4}\right) = 1.0.$$

[redacted]

[redacted]

$$\frac{1}{\psi} = 100 \frac{L^2}{K} \sqrt{2} = 141 \frac{L^2}{K}$$

$K = s c r_g$

$s = 1.0$

$c = 12,000$ fps

$r_g = 0.2\, D_c \sqrt{\sigma} = 0.2 \times 4 \times \sqrt{1.2} \cong 0.88$

$K = 1 \times 0.88 \times 12,000 \cong 10,500$ ft²/sec.

$$\frac{1}{\psi} = 141 \left(\frac{20 \times 20}{10,500}\right) = 5.4 \text{ sec}$$

$$\gamma = \psi\, (W(Mt))^{1/3} \left(\frac{100 \text{ psi}}{p_{so}}\right)^{7/8}$$

$$= \left(\frac{1}{5.4}\right)\left(\frac{0.05}{1}\right)^{1/3}\left(\frac{100}{200}\right)^{7/8} = 0.04$$

$$\gamma' = \left(\frac{H_{av}}{370 \text{ ft}}\right)\left(\frac{1}{W(Mt)}\right)^{1/3}\left(\frac{1,000 \text{ fps}}{c_p}\right)\left(\frac{p_{so}}{100 \text{ psi}}\right)^{1/2}$$

$$= \left(\frac{5}{370}\right)\left(\frac{1}{0.05}\right)^{1/3}\left(\frac{1,000}{2,500}\right)\left(\frac{200}{100}\right)^{1/2}$$

$\gamma' = 0.02$

From Figure 11-32 $t_d/T = 0.9$

Step (4):

From Figure 11-30

$$\frac{p_d}{q_y X} = 2.0 \text{ (Severe Damage)}$$

Step (5):

$$p_d = 2.0\, q_y X = (2.0)(220)(1) = 440 \text{ psi}$$

Step (6):

$$\alpha = \left(\frac{H_{av}}{370 \text{ ft}}\right)\left(\frac{1}{W(Mt)}\right)^{1/3}\left(\frac{1,000 \text{ fps}}{c_p}\right)$$

$$= \left(\frac{5}{370}\right)\left(\frac{1}{0.05}\right)^{1/3}\left(\frac{1,000}{2,500}\right) = 0.015$$

From Figure 11-29 $\beta \cong 1$ and it does not depend on p_{so} very markedly.

Step (7):

$$p_{so} = \beta\, p_d = (1)(440) = 440 \text{ psi}$$

which is greater than the 200 psi assumed; therefore, the required number of Steps (3) through (7) must be repeated (see paragraph 11-8).

Step (3):

Assume $p_{so} = 1,000$ psi (increased because of large difference between the assumed and the final value obtained above.)

$\frac{1}{\psi} = 5.4$ sec since it depends on structural properties only.

$$\gamma = \left(\frac{1}{5.4}\right)\left(\frac{0.05}{1}\right)^{1/3}\left(\frac{100}{1,000}\right)^{7/8} = 0.009$$

$$\gamma' = \left(\frac{5}{370}\right)\left(\frac{1}{0.05}\right)^{1/3}\left(\frac{1,000}{2,500}\right)\left(\frac{1,000}{100}\right)^{1/2} = 0.046$$

From Figure 11-32.

$$\frac{t_d}{T} < 0.3;$$

therefore, the assumed loading is impulsive.

[redacted]

[redacted]

[redacted]

Step (4).

From Figure 11-33.

$$\delta = c\,(W/Mu)^{1/3} = \left(\frac{1}{5.4}\right)\left(\frac{0.05}{1}\right)^{1/3} = 0.07$$

$$q_y X = (220)(1) = 220 \text{ psi.}$$

from previous calculation. From Figure 11-34b, the values of δ and $q_y X$ would intersect below and to the right of the limiting line. However, from Figure 11-32, it is known that $t_d/T < 0.3$; thus, the limiting curve must be used. For

$$q_y X = 220 \text{ psi.}$$

$$p_{so} = 1{,}100 \text{ psi}$$

on the limiting curve. Since this is greater than the 1,000 psi assumed, it must be investigated to determine if 1,100 psi also corresponds to impulsive conditions (i.e., $t_d/T < 0.3$ as defined herein). Therefore, recompute as in Step (3).

For p_{so} = 1,100 psi:

$$\gamma = \left(\frac{1}{5.4}\right)\left(\frac{0.05}{1}\right)^{1/3}\left(\frac{100}{1{,}100}\right)^{7/8} = 0.008$$

$$\gamma' = \left(\frac{5}{370}\right)\left(\frac{1}{0.05}\right)^{1/3}\left(\frac{1{,}000}{2{,}500}\right)\left(\frac{1{,}000}{100}\right)^{1/2} = 0.046$$

From Figure 11-32,

$$t_d/T < 0.3$$

Answer: The correct side-on overpressure at the surface to produce severe damage to the roof of the buried rectangular, reinforced concrete structure is

$$p_{so} = 1{,}100 \text{ psi.}$$

(NOTE: If the wall rather than the roof of the structure were being investigated, the value of $q_y X$ (if Figure 11-34 controls) or p_d (if Figure 11-30 controls) would be divided by the appropriate value of K_o from Table 11-6 to find the value of p_{so} or of p_y, respectively.)

Related Material: See paragraph 11-8. See also Sections I and II of Chapter 2.

[redacted]

Problem 11-5 Calculation of the Thickness Required for Shallow Buried Reinforced Concrete Arch

[redacted] Figures 11-29 through 11-43 and Tables 11-6, 11-7, and 11-10 provide the information necessary to determine approximate dimensions of deeply buried structures; however, as described in paragraph 11-9, these figures and tables, together with Figure 11-44, also may be used to analyze structural dimensions of shallow buried buildings.

[redacted] *Example* [redacted]

Given: A semi-circular reinforced concrete arch to be buried with the crown level with the surface of the earth, i.e., depth of cover over the crown $H_c = 0$. Other pertinent characteristics of the structure and the medium are:

Required span length, $L = 60$ feet
Arch radius, $r = 30$ feet
Effective seismic velocity of the medium, $c_p = 1{,}000$ fps.

Find: The required thickness of the concrete that will allow the structure to sustain no more than moderate damage from a 5 Mt burst at a distance where the peak side-on overpressure, p_{so}, is 200 psi at the surface of the ground.

Solution: Since the arch is semi-circular,

$$h = r = 30 \text{ feet}$$

and

$$\frac{h}{L} = \frac{30}{60} = 0.5.$$

From Figure 11-26,

$$\frac{H_{av}}{L} = 0.11.$$

Therefore,

$$H_{av} = 0.11L = 0.11 \times 60 = 6.6 \text{ feet}.$$

A value of

$$q_y = p_{so} = 400 \text{ psi}$$

is assumed in order to obtain a trial thickness from the curves for deeply buried structures. Using this value of q_y to enter Figure 11-35 gives

$$\frac{D_c}{r} = 0.84$$

$$D_c = 0.84r = 0.84 \times 30 = 2.5 \text{ feet}.$$

From paragraph 11-9, the effective span b_s of an arch buried at shallow depth for use in determining the resistance to flexure alone, i.e., for use with Figure 11-38, is one half the developed length of the arch. Therefore,

$$b_s = \frac{2\pi r}{4} = 1.57 \times 30 = 47 \text{ feet}$$

and

$$\frac{D_c}{b_s} = \frac{2.5}{47} = 0.053.$$

Since $p_f f_y$ is unknown, a value of 600 is used in Figure 11-38, and from that figure

$$q_y = 25 \text{ psi}$$

for restrained edges. For simply supported edges

[redacted]

which exist in this mode of response as defined in Figure 11-38

$$q_y = 1/2 \times 25 \cong 13 \text{ psi}.$$

Since the required flexural resistance is of the order of $1/2\, p_{so}$ = 100 psi, the thickness of 2.5 feet obtained as a trial value above probably is not sufficient. Therefore, a new value is assumed that is approximately twice as large, i.e., D_c = 4.5 feet. From Figure 11-35, q_y for a deeply buried arch is about 720 psi; call this q_a. From Figure 11-38, with D_c/b_y = 0.096, q_y for a deeply buried simply supported, one-way slab = $1/2 \times 80$ = 40 psi; call this q_b. If the effect of loading duration is ignored for the moment by setting p_{so}/q_y = 1.0 for either mode of loading, the adequacy of an arch of 4.5 foot thickness may be investigated tentatively. (Note that according to paragraph 11-9, attenuation with depth for shallow burial is so small that it can be ignored). Let $(p_{so})_a$ be the component of overpressure producing thrust in the arch. From paragraph 11-9, $(p_{so})_a = p_{so}$ = 200 psi. Similarly, the component of overpressure producing flexure in the arch is $(p_{so})_b = 1/2\, p_{so}$ = 100 psi. The ratios P/P_u and M/M_p for entering Figure 11-44 must now be calculated. These are, for the conditions above:

$$P/P_u = \frac{(p_{so})_a}{q_a} = \frac{200}{720} = 0.28$$

$$M/M_p = \frac{(p_{so})_b}{q_b} = \frac{100}{40} = 2.5$$

From Figure 11-44, with these values, and for $p_f\, f_y/f'_c$ = 600/4,000 = 0.15 ($p_f\, f_y$ = 600 psi from above and f'_c = 4,000 psi assumed).

$$P/P_u = 0.28.$$

$$M/M_p = 1.8.$$

This is close enough to the value of 2.5 above to proceed. Now investigate the effect of loading duration that was temporarily ignored above.

From Table 11-10, for the thrust component of loading:

$$\psi = \left(\frac{10 \text{ ft}}{r}\right) \frac{c_m}{1{,}000 \text{ fps}}$$

$$\text{and } c_m = 1{,}800 \text{ fps}.$$

$$\psi = \left(\frac{10}{30}\right)\left(\frac{1{,}800}{1{,}000}\right) = 0.6$$

$$\gamma = \psi\, (W/(Mt))^{1/3} \left(\frac{100 \text{ psi}}{p_{so}}\right)^{7/8}$$

$$= (0.6)\left(\frac{5}{1}\right)^{1/3} \left(\frac{100}{200}\right)^{7/8} = 0.56$$

$$\gamma' = \frac{H_{av}}{370 \text{ ft}} \left(\frac{1}{W/(Mt)}\right)^{1/3} \left(\frac{1{,}000 \text{ fps}}{c}\right)\left(\frac{p_{so}}{100 \text{ psi}}\right)^{1/2}$$

$$= \left(\frac{6.6}{370}\right)\left(\frac{1}{5}\right)^{1/3} \left(\frac{1{,}000}{1{,}000}\right)\left(\frac{200}{100}\right)^{1/2}$$

$$\gamma' = 0.01$$

From Figure 11-32, $t_d/T > 3$; therefore p_{so}/q_y = 1.

From paragraph 11-10 for the flexural component of loading:

$$\gamma = \psi \left(\frac{10 \text{ ft}}{L}\right)\left(\frac{U}{1{,}000 \text{ fps}}\right)$$

From paragraph 2-15, Chapter 2:

$$U = 1{,}116 \text{ fps} \sqrt{1 + \frac{6\, p_{so}}{7\, P_o}}$$

[redacted]

$$= 1.116 \sqrt{1 + \frac{6 \times 200}{7 \times 14.7}} = 4.000 \text{ fps}$$

$$\gamma = \psi\left(\frac{10}{60}\right)\left(\frac{4.000}{1.000}\right) = 0.67\psi$$

From Table 11-10.

$$\frac{1}{\psi} = 100 \frac{L^2}{K} \sqrt{1 + \Gamma} \quad (L = b_s)$$

$$K = s c_g = 1 \times 12.000 \times 0.2$$

$$\times 4.5 \times \sqrt{\frac{600 \times 100}{40.000}}$$

(p_i f'_c = 600 psi from above and f_y = 40.000 psi assumed)

$$K = 13.000 \text{ ft}^2/\text{sec.}$$

$$\frac{1}{\psi} = 100 \left(\frac{47 \times 47}{13.000}\right) \sqrt{1 + 0.8 \frac{6.6}{5}} = 24$$

$$\gamma = \frac{0.67}{24} = 0.028$$

$\gamma' = 0.01$ from first trial above.

From Figure 11-32.

$$t_d/T = 0.6$$

From Figure 11-30.

$$p_d/q_y X = 1.5 \text{ (Moderate Damage)}$$

Since $X = \alpha = 1$ for shallow burial and $p_d = p_{so}$ 2 (see paragraph 11-9).

$q_y = \frac{100}{1.5} = 70$ psi; call this last value q'_b.

$$\frac{q'_b}{q_b} = \frac{70}{40} = 1.8$$

Answer: Since this value agrees with the value obtained above from Figure 11-44, an arch 4.5 ft thick, with 1.5 percent reinforcement, each face and with f'_c = 4,000 psi and f_y = 40,000 psi is adequate for the given conditions.

(NOTE: Because an "analysis" the thickness D_c of the structure is known, analysis follows directly as given in the preceding example using the known value of D_c instead of the assumed value. In analysis, however, p_{so}, the side-on overpressure is unknown; thus a value must be assumed and the steps above are followed to find a new value of overpressure to compare with that assumed.)

Related Material: See paragraph 11-9. See also Sections I and III, Chapter 2 and Appendix B.

[redacted]

Problem 11-6 Calculation of the Vulnerability of the Near Surface Portion of a Reinforced Concrete Silo

[redacted] Figures 11-29 through 11-44 and Tables 11-6, 11-7, and 11-10 may be used to evaluate the vulnerability of silos buried in soil. Only the procedures that apply to the near-surface portion (above a depth equal to approximately the diameter) of a cylindrical silo are illustrated below. At greater depths, the procedure is straightforward; the direct circumferential thrust is the controlling force, so the circumferential reinforcement dominates, but additional longitudinal reinforcement may be required to resist skin friction.

[redacted] In the following example, it is assumed [redacted] the cover of the silo is supported directly on the structure. However, the longitudinal stress and the stiffening effect of the cover or the diaphragm, which frequently is associated with the cover, are neglected in the following. The longitudinal stress will normally require at least consideration of added longitudinal reinforcement. Neglect of the stiffening effect in the following results in an overpressure that is less than that which would cause the desired damage, that is, a conservative result from the standpoint of defense.

[redacted] *Example* [redacted]

Given: A reinforced concrete silo buried in loose dry sand with the cover flush with the surface of the earth. Characteristics of the silo and the medium are:

[redacted]

DNA (b)(1)

[redacted]

DNA (b)(1)

Find: The side-on overpressure at surface that will cause no more than moderate damage to the silo when delivered by a 5 Mt surface burst.

[redacted]

DNA (b)(1)

Pages 11-69 + 11-70 are deleted.

[redacted]

[redacted]

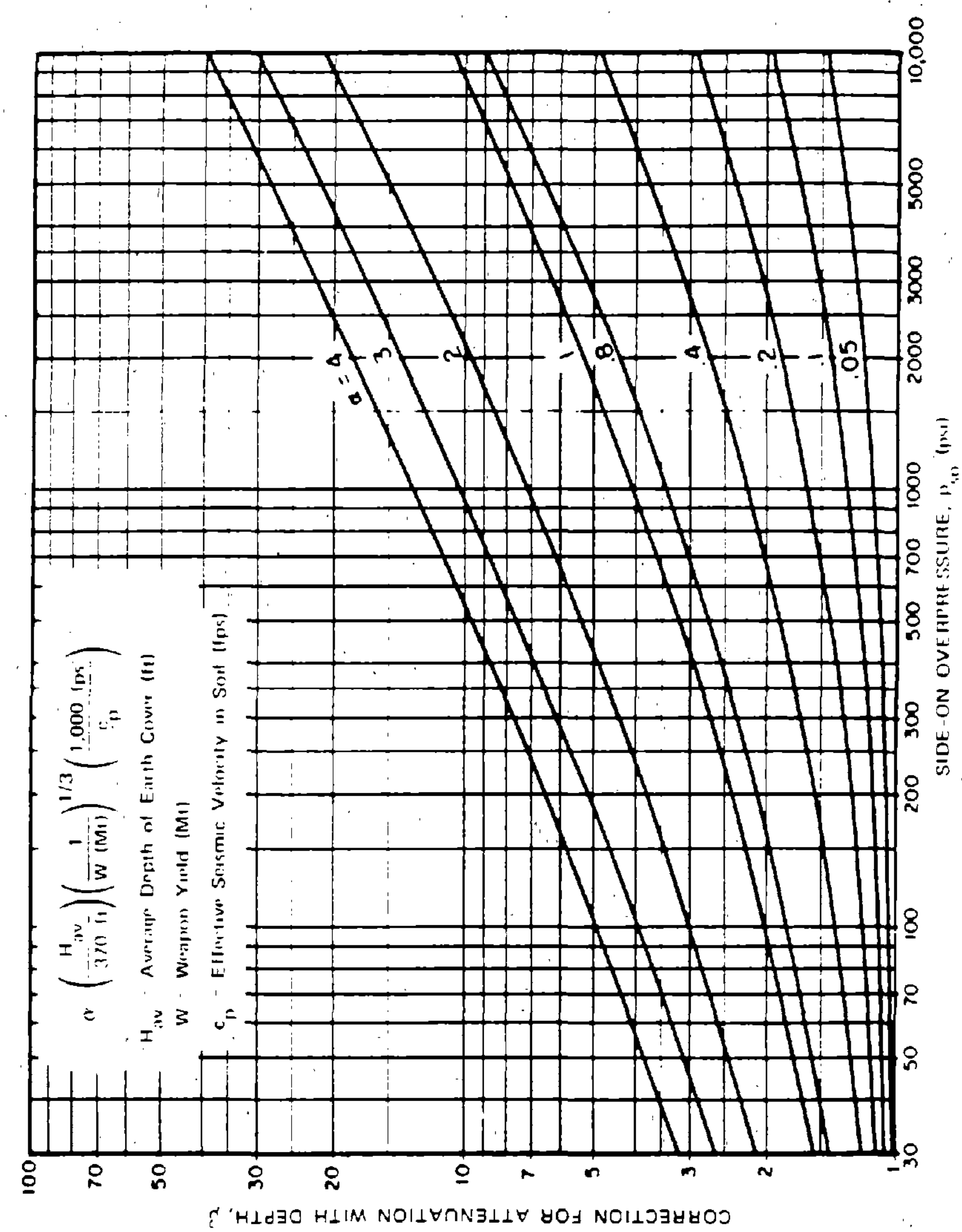


Figure 11-29. [redacted] Correction for Attenuation with Depth [redacted]

[redacted]

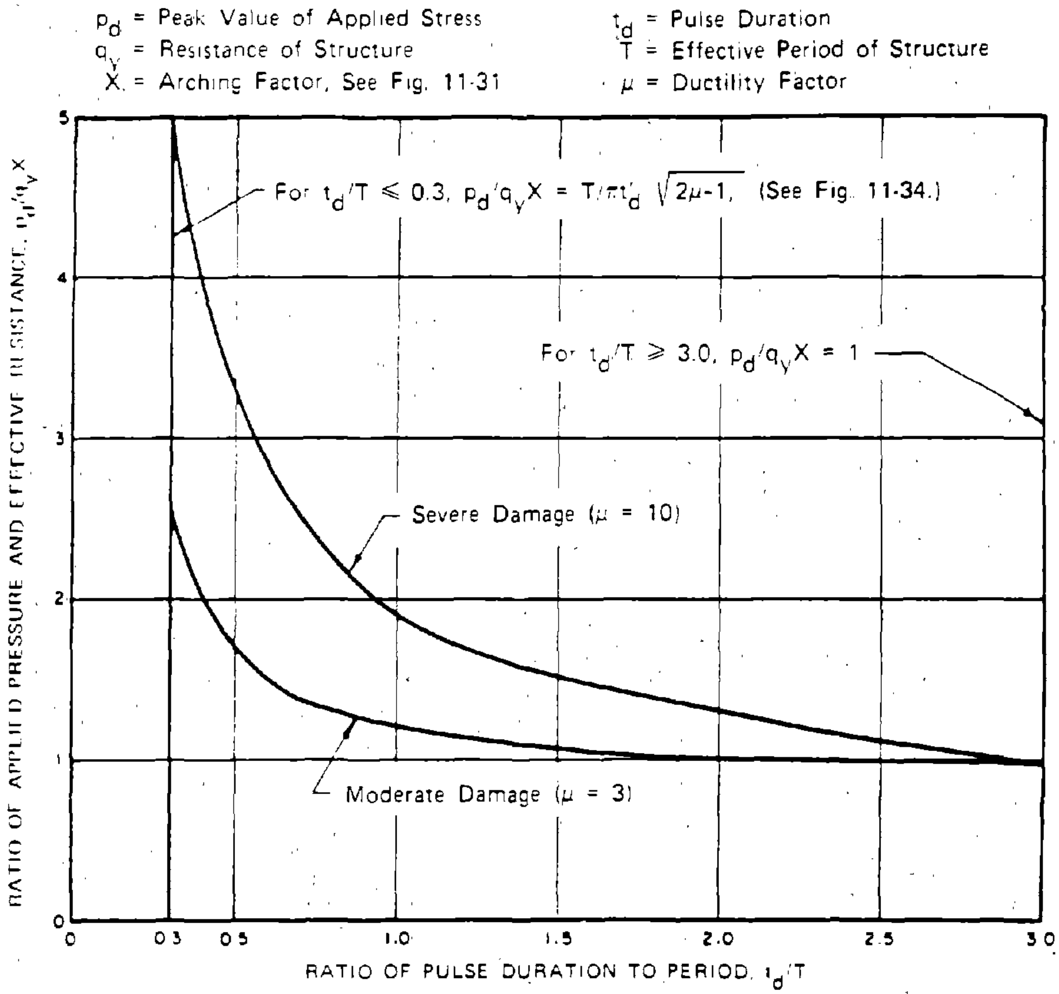


Figure 11-30. [redacted] Damage Pressure Level [redacted]

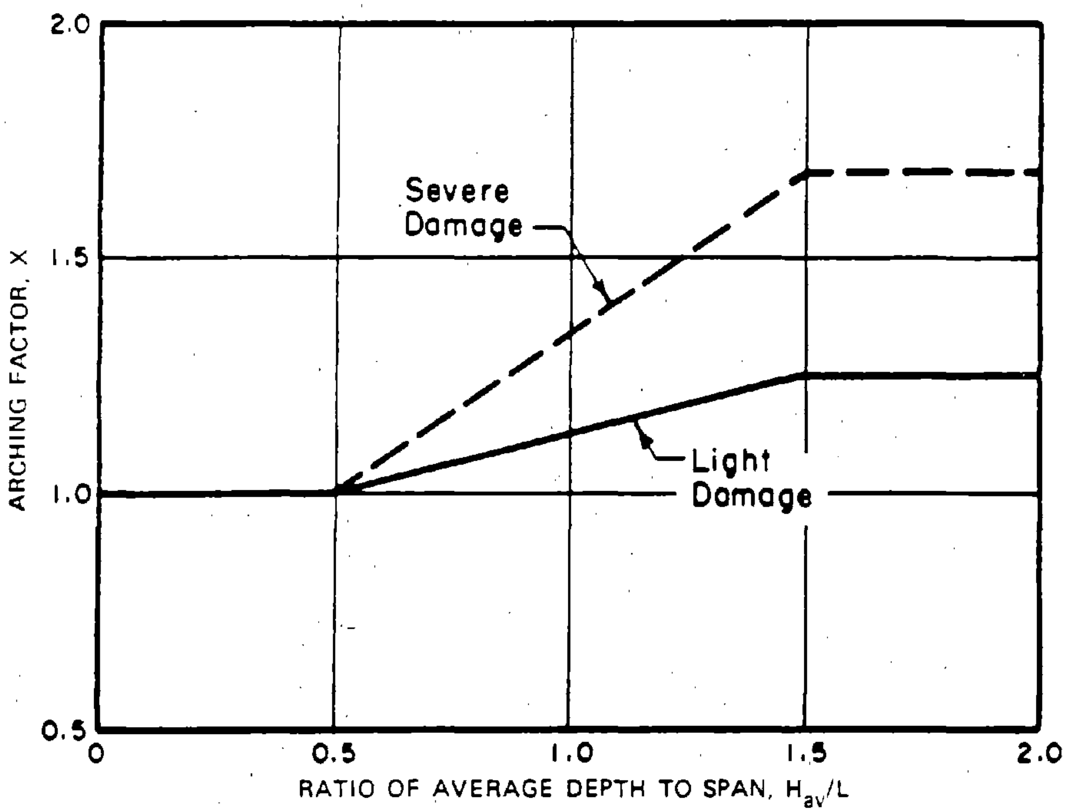
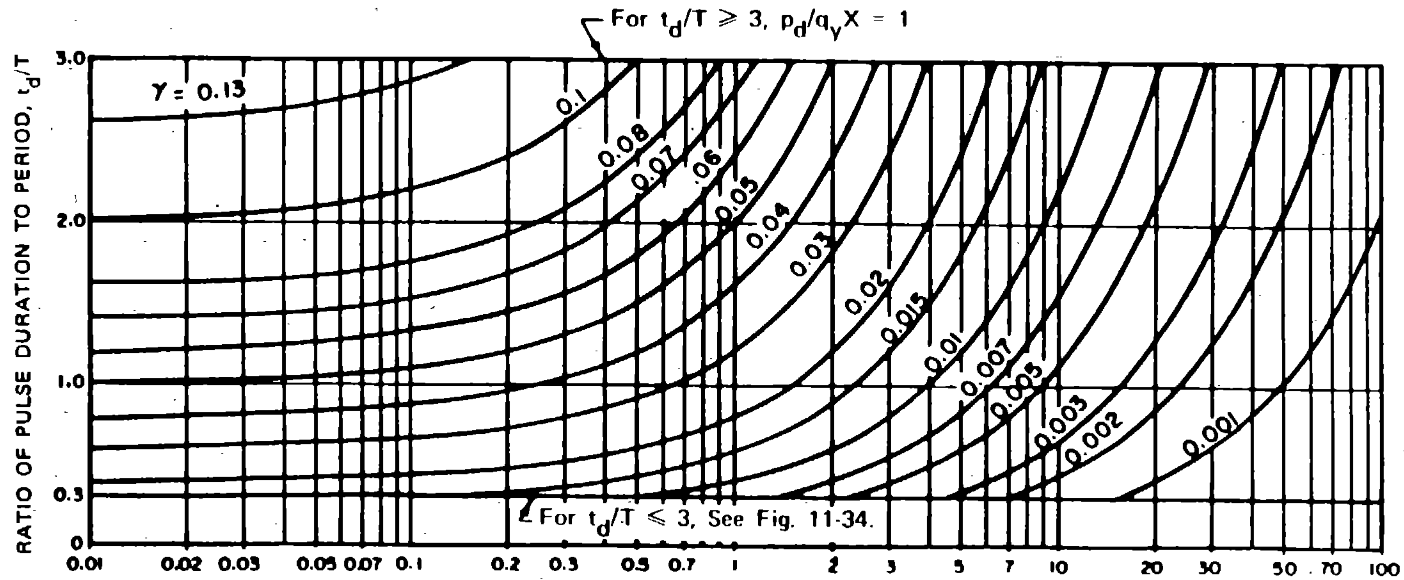


Figure 11-31. [redacted] Correction for Arching [redacted]

[redacted]

For Definition of γ See Table 11-10. Other Parameters are Defined in Preceding Figures.



$$\gamma' = \left(\frac{H_{av}}{370\ \text{ft}}\right)\left(\frac{1}{W\ (\text{Mt})}\right)^{1/3}\left(\frac{1{,}000\ \text{fps}}{c_p}\right)\left(\frac{p_{so}}{100\ \text{psi}}\right)^{1/2}$$

Figure 11-32. [redacted] Definition of Effective Pulse Duration [redacted]

[redacted]

[redacted]

Table 11-10. [redacted] Definition of γ and δ for Figures 11-32 and 11-34 [redacted]

$$\gamma = \psi\ (W(Mt))^{1/3} \left(\frac{100\ \text{psi}}{P_{so}}\right)^{7/8} \qquad \delta = \psi\ (W(Mt))^{1/3}$$

ψ is defined by following:

For arches and domes:

$$\psi = \left(\frac{10\ \text{ft}}{r}\right)\left(\frac{c_m}{1{,}000\ \text{fps}}\right)$$

c_m (fps)	Structural Type
1,800	Concrete arch or cylinder
2,700	Steel or aluminum arch or cylinder concrete dome
3,800	Steel or aluminum dome

For one-way slabs:

$$\frac{1}{\psi} = 100\,\frac{L^2}{K}\sqrt{1 + \Gamma}\ ,$$ where L = span of structure, and

Γ (defined by structural type)	Structural Type
$0.8\,\frac{L}{D_c}$	Reinforced concrete walls
$0.8\,\frac{H_{av}}{D_c}$ $\begin{cases} \text{for } H_{av} \leq L \\ \text{for } H_{av} > L\text{, use} \\ H_{av} = L \end{cases}$	Reinforced concrete roofs
0.2 [redacted] $\frac{Lx}{A}$	Steel walls (x = spacing of members)

[redacted]

Table 11-10 [redacted] (Concluded)

Γ (defined by structural type)	Structural Type
$0.2 \frac{H_{av} x}{A} \begin{cases} \text{for } H_{av} \leqslant L \\ \text{for } H_{av} > L\text{, use} \\ H_{av} = L \end{cases}$	Steel roofs
$0.8 \frac{Lx}{A}$	Aluminum walls
$0.8 \frac{H_{av} x}{A} \begin{cases} \text{for } H_{av} \leqslant L \\ \text{for } H_{av} > L\text{, use} \\ H_{av} = L \end{cases}$	Aluminum roofs

$K = scr_g$

where: c = seismic velocity of material:
20,000 fps for steel or aluminum
12,000 fps for reinforced concrete

r_g = radius of gyration of element
(Use $r_g = 0.2D_c \sqrt{\phi}$ for reinforced concrete ($\phi = 100\ p_f$ and D_c in ft);
See Figure 11-33 for structural shapes)

s	Support Condition for Element
1	1-way slab, simple support each end
2	1-way slab, fixed support each end
1.5	1-way slab, fixed one end; simple other end
0.3	1-way slab, cantilevered span
$1 + (b_s/b_L)^2$	2-way slab, simple support on all edges
$0.6 \left[1 + (b_s/b_L)^2\right]$	2-way slab, fixed support on all edges
1.7	Square, flat slab

J = 1/2 Plastic Moment Arm, in.

r_g = Radius of Gyration, in.

D_s = Depth of Section, in.

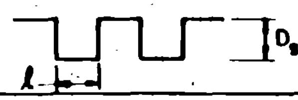
SECTION		$\frac{J}{D_s}$	$\frac{r_g}{D_s}$
SOLID PLATES		0.25	0.29
CORRUGATED METAL		0.30	0.34
CORRUGATED METAL		$\dfrac{\frac{1}{2}\frac{l}{D_s}+\frac{1}{4}}{1+\frac{l}{D_s}}$	$\sqrt{\dfrac{\frac{1}{4}\frac{l}{D_s}+\frac{1}{12}}{1+\frac{l}{D_s}}}$
ROLLED SHAPES		0.39	0.42
ROLLED SHAPES WITH COVER PLATES		0.41	0.43
ROLLED SHAPES WITH ONE COVER PLATE		0.46	0.41

Figure 11-33. [redacted] Approximate Values of J and r_g for Common Shapes [redacted]

[redacted]

For Definition of δ, See Table 11-10. Other Parameters are Defined in Preceding Figures.

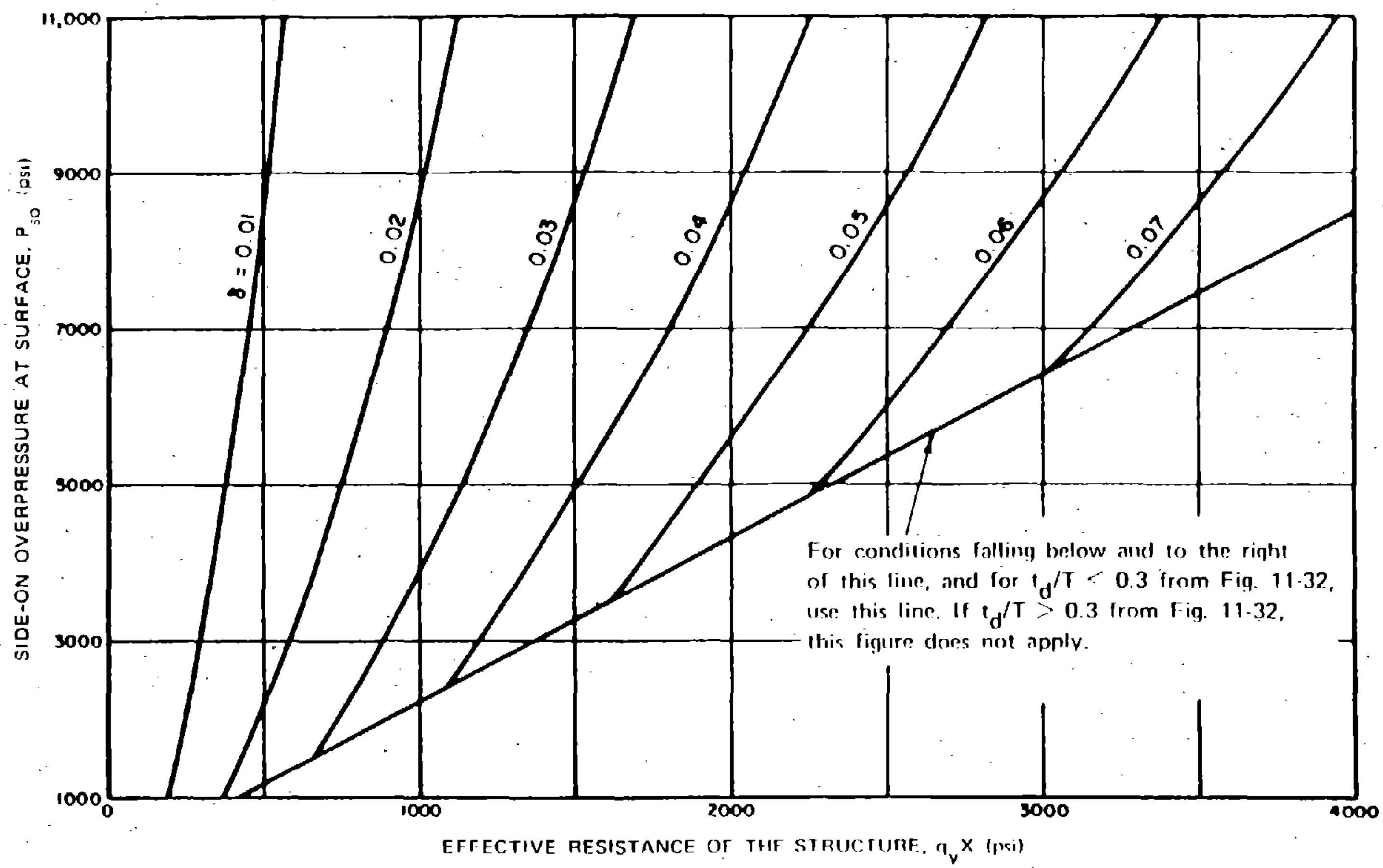


Figure 11-34a. [redacted] Conditions for Moderate Damage by Impulsive Loading [redacted]

[redacted]

[redacted]

For Definition of δ, See Table 11-10. Other Parameters are Defined in Preceding Figures.

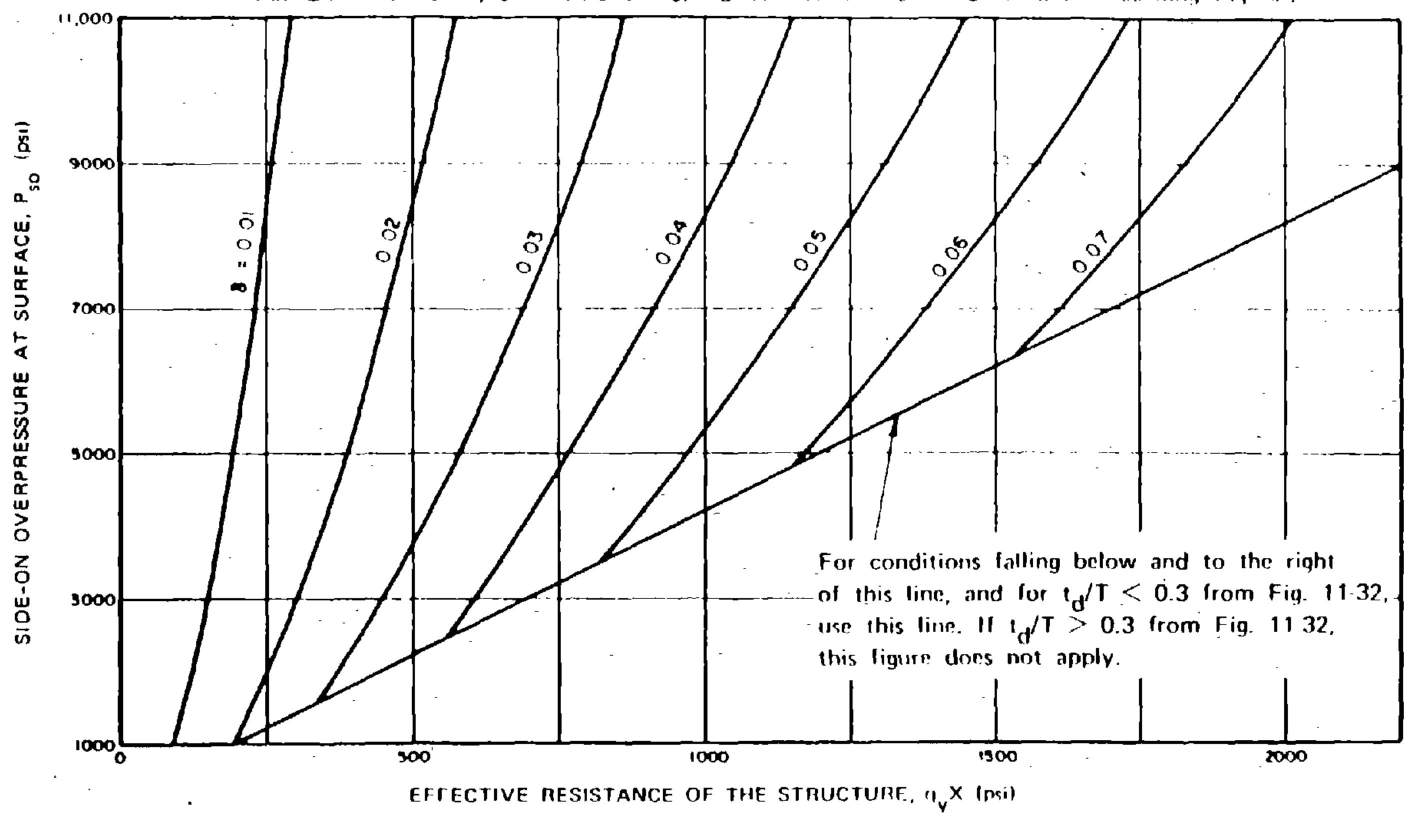


Figure 11-34b. [redacted] Conditions for Heavy Damage by Impulsive Loading [redacted]

[redacted]

[redacted]

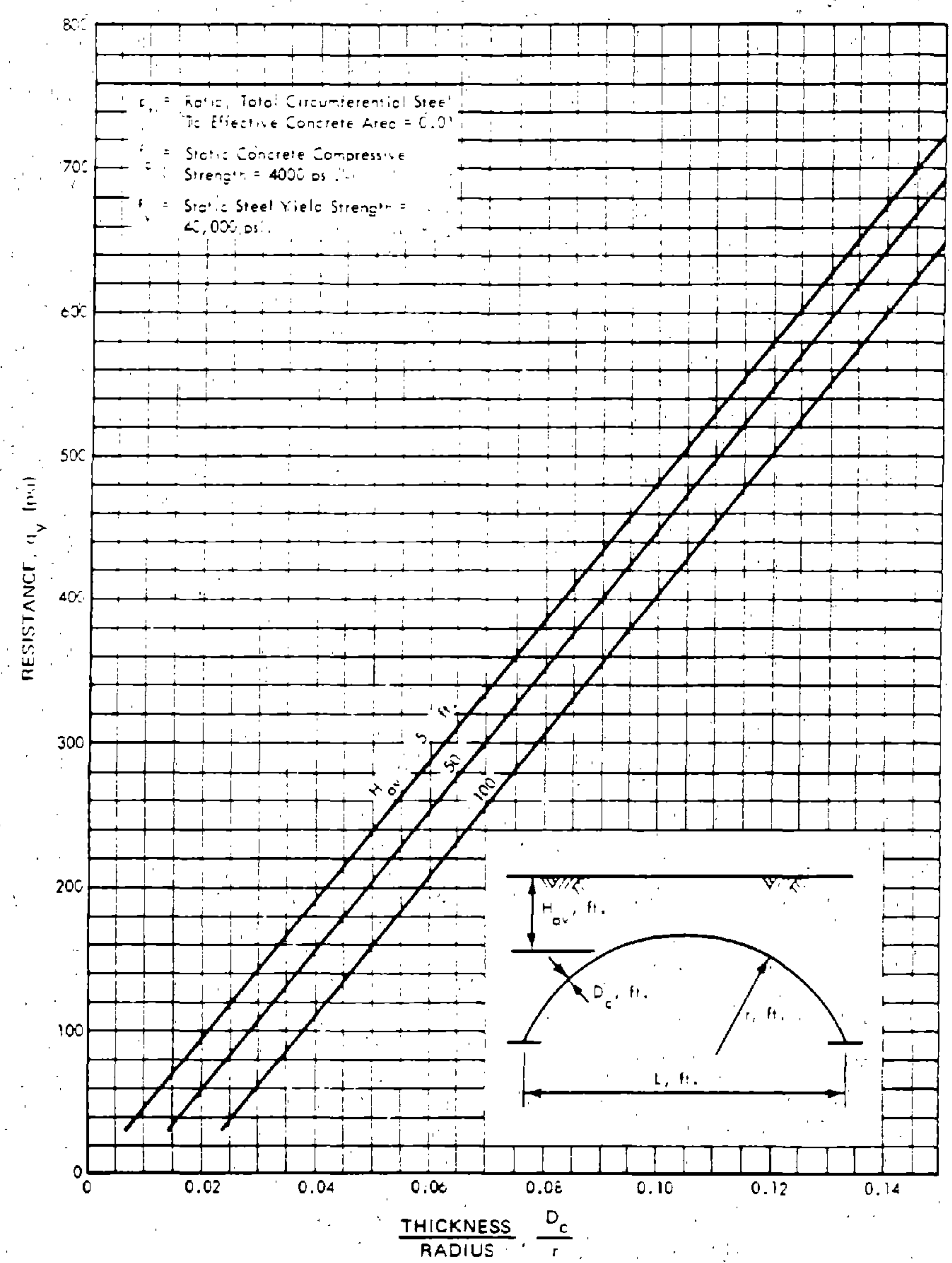


Figure 11-35. [redacted] Resistance of Deeply Buried Horizontal Reinforced Concrete Arches [redacted]

[redacted]

[redacted]

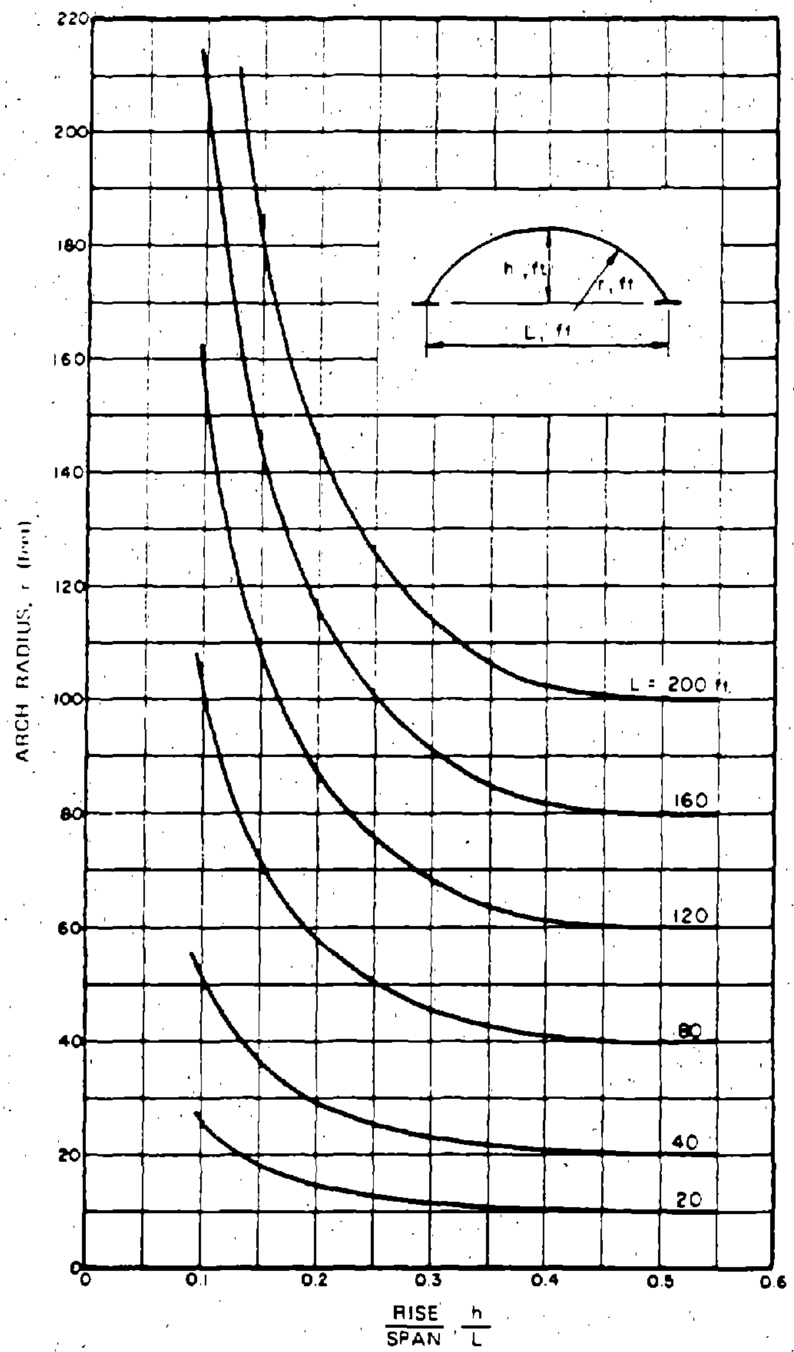


Figure 11-35a. [redacted] Arch Radius Versus Rise-Span Ratio [redacted]

[redacted]

[redacted]

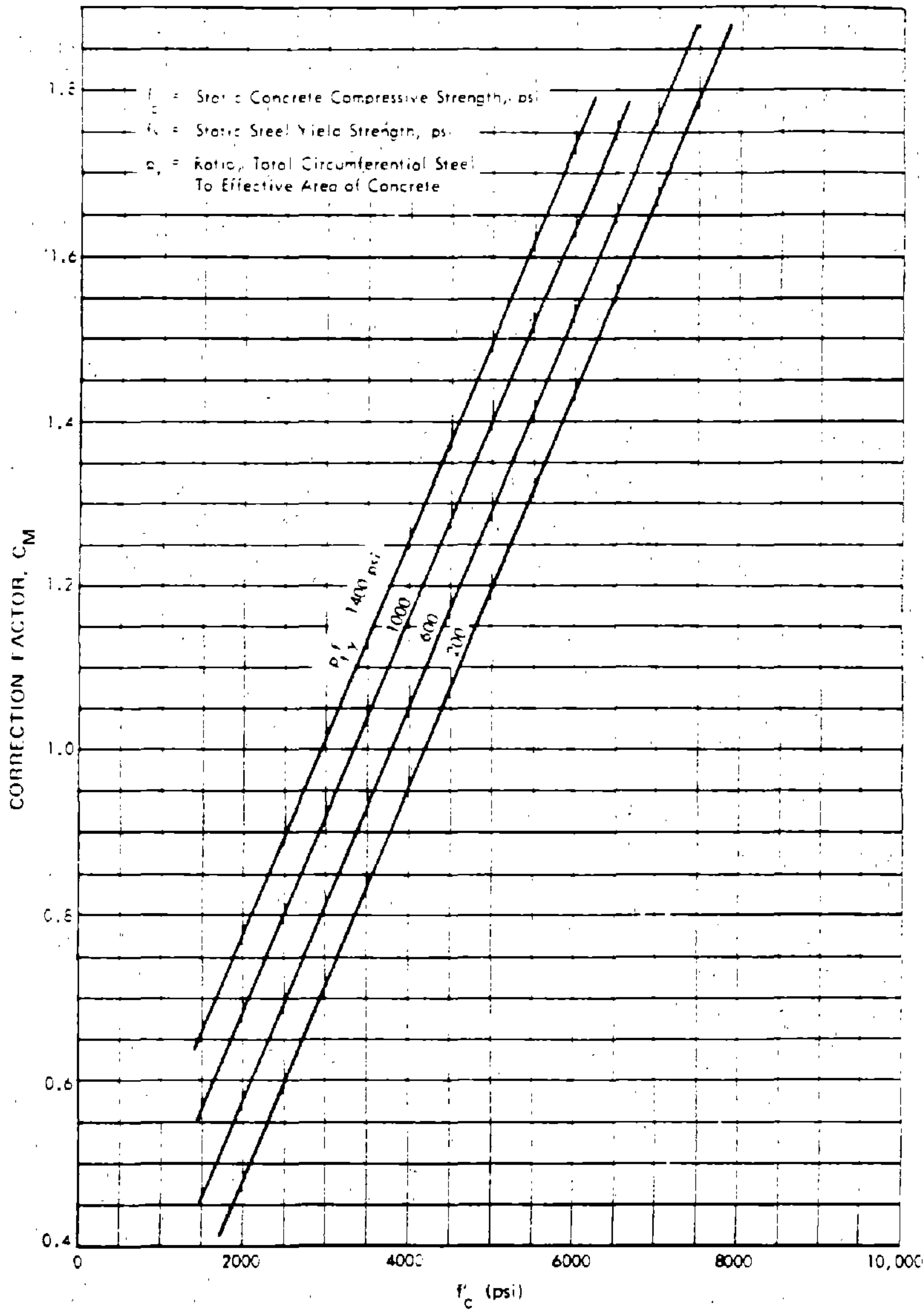


Figure 11-35b. [redacted] Correction Factor for Material Properties for Deeply Buried Horizontal Reinforced Concrete Arches [redacted]

[redacted]

[redacted]

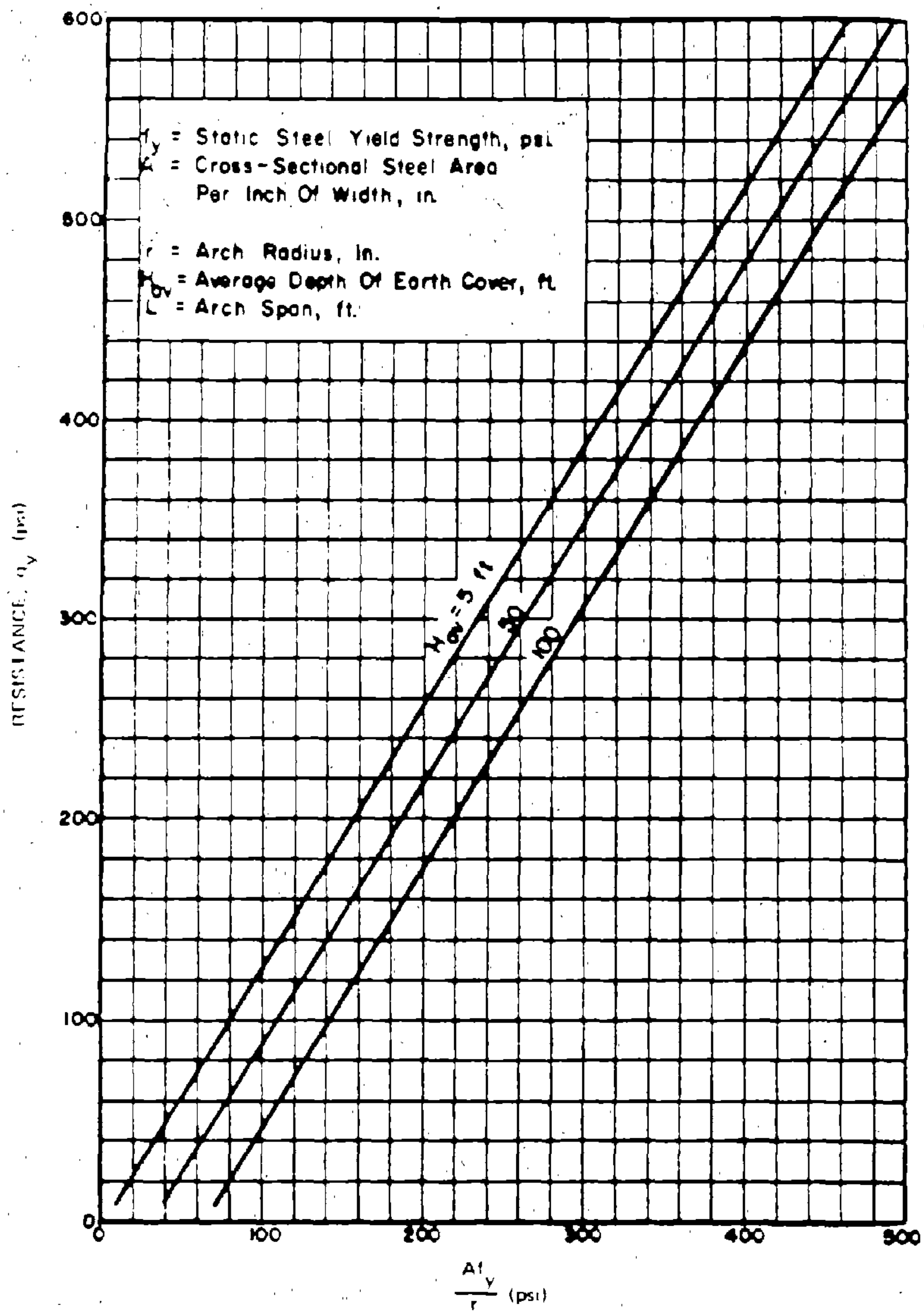


Figure 11-36. [redacted] Resistance of Deeply Buried Horizontal Steel Arches [redacted]

[redacted]

[redacted]

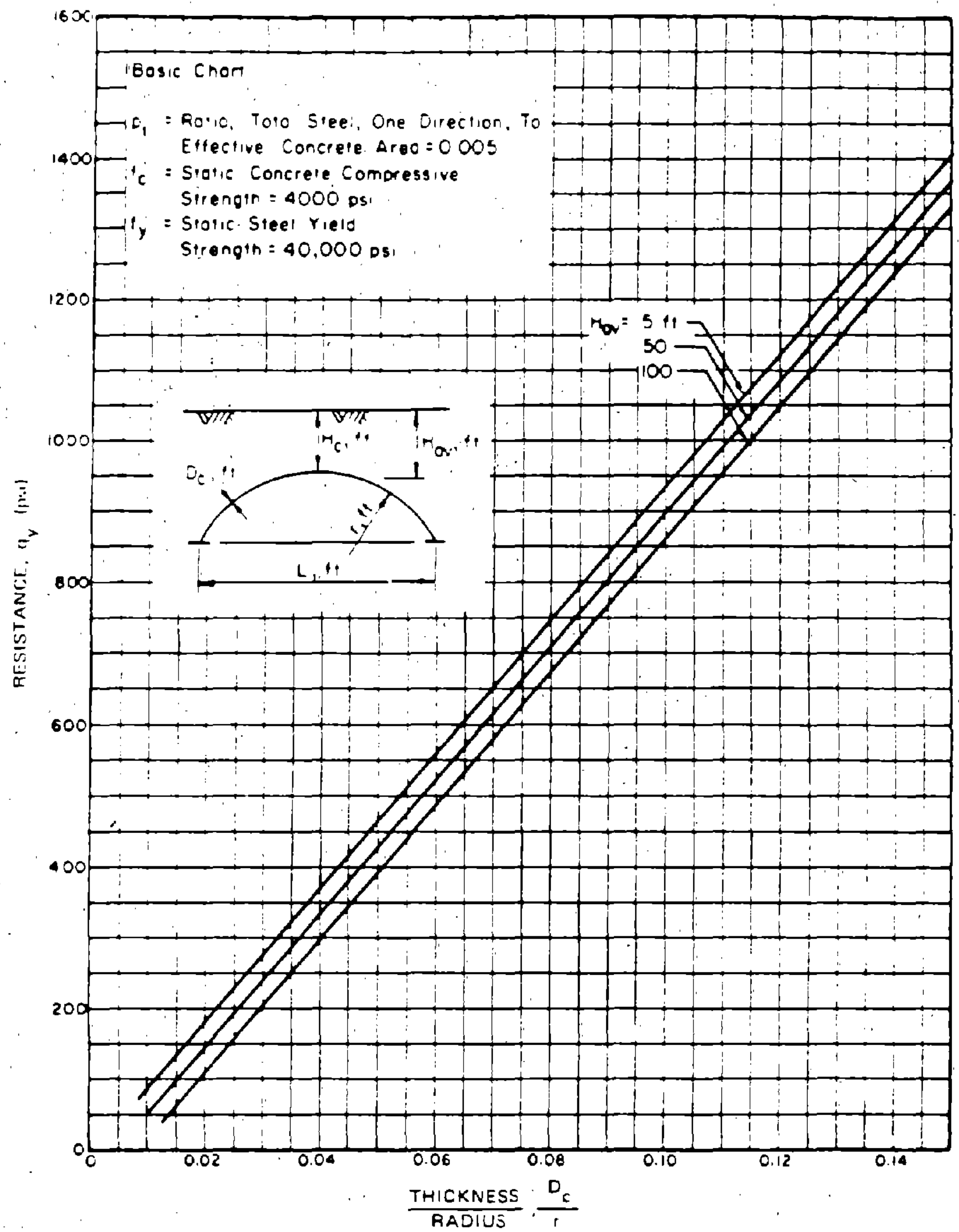


Figure 11-37. [redacted] Resistance of Deeply Buried Reinforced Concrete Domes [redacted]

[redacted]

[redacted]

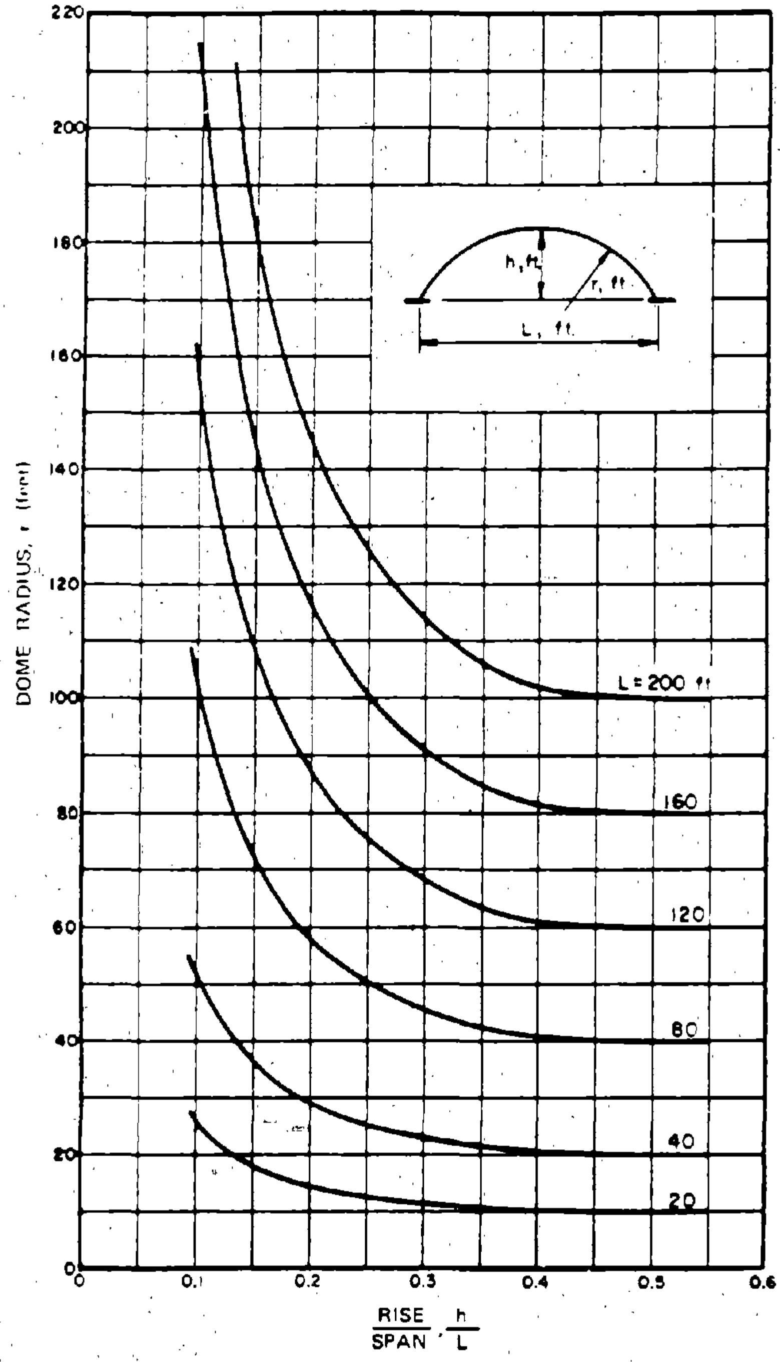


Figure 11-37a. [redacted] Dome Radius Versus Rise-Span Ratio [redacted]

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[redacted]

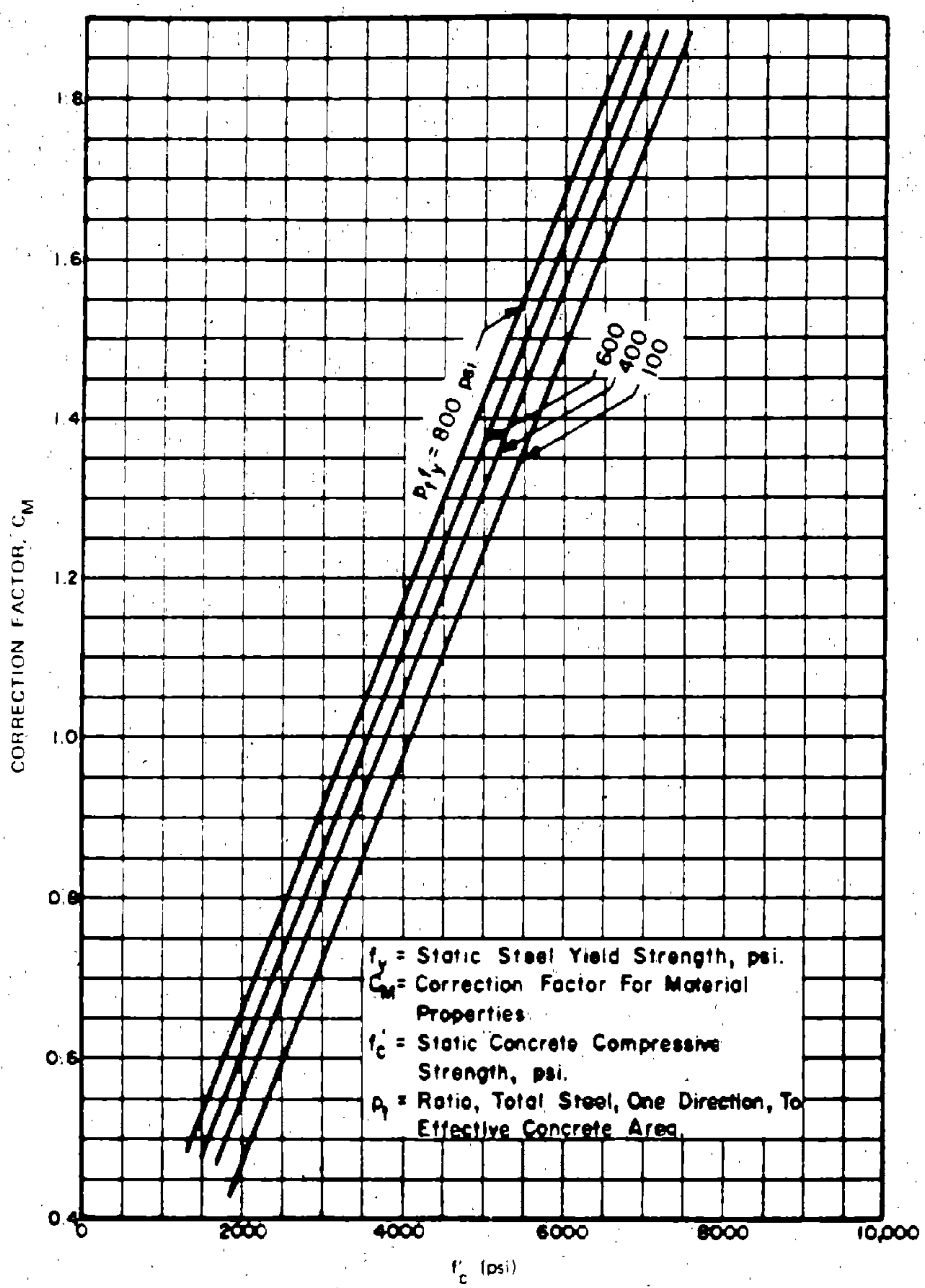

Figure 11-37b. [redacted] Correction Factor for Material Properties for Deeply Buried Reinforced Concrete Domes [redacted]

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[redacted]

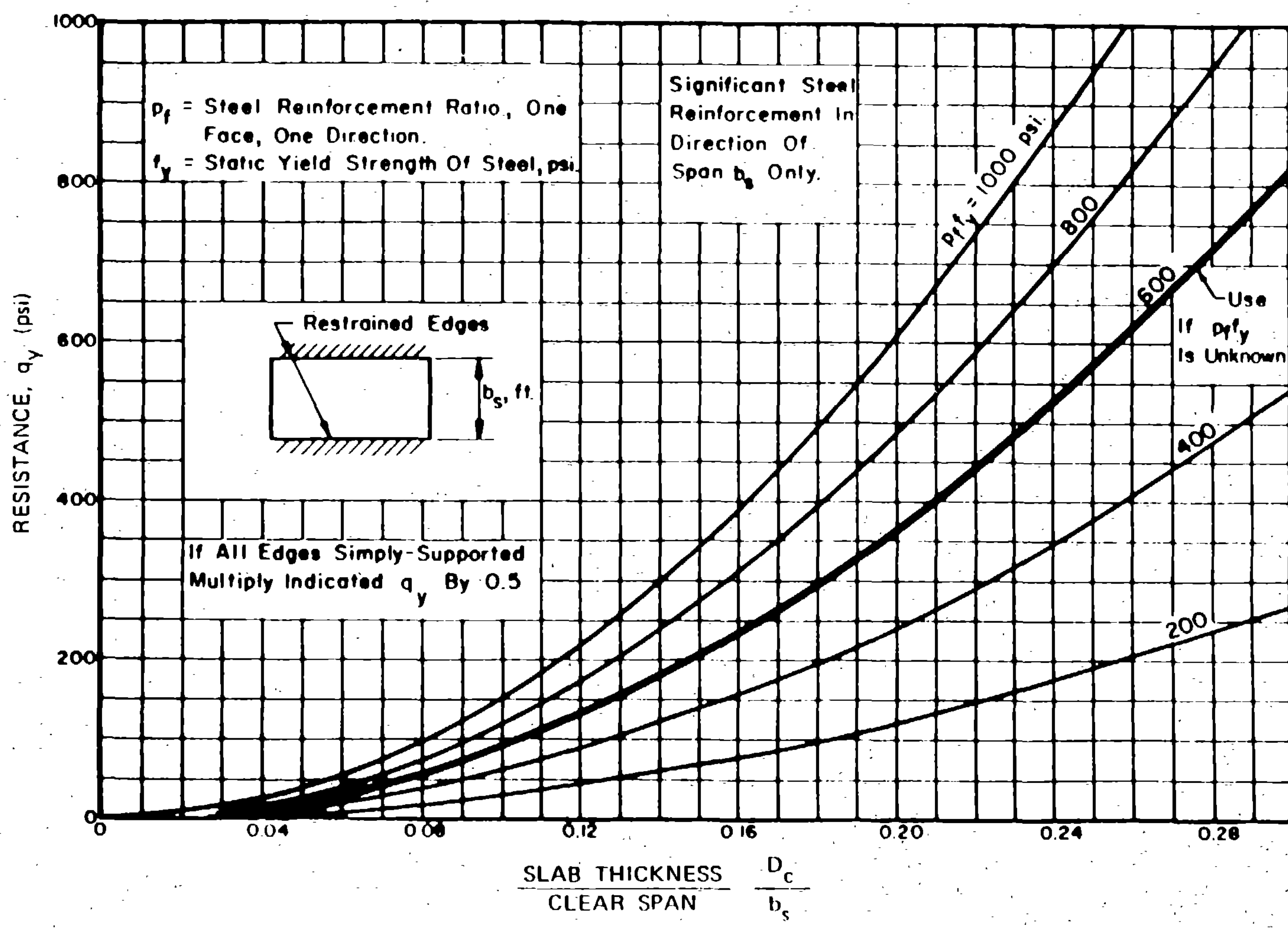


Figure 11-38. [redacted] Resistance of Horizontal One-Way Slabs [redacted]

[redacted]

[redacted]

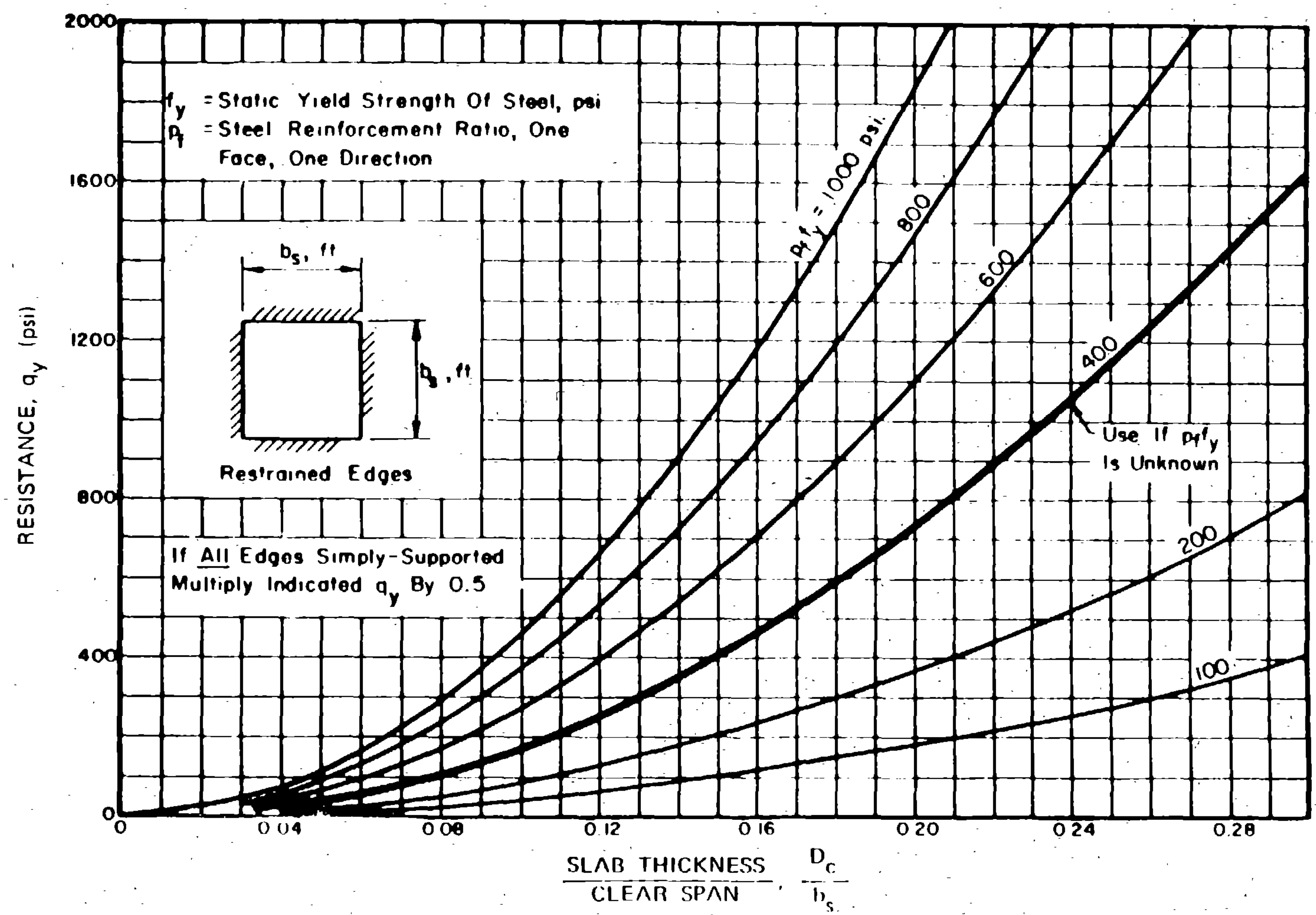


Figure 11.39. [redacted] Resistance of Horizontal Square Two-Way Slabs — Restrained Edges [redacted]

[redacted]

[redacted]

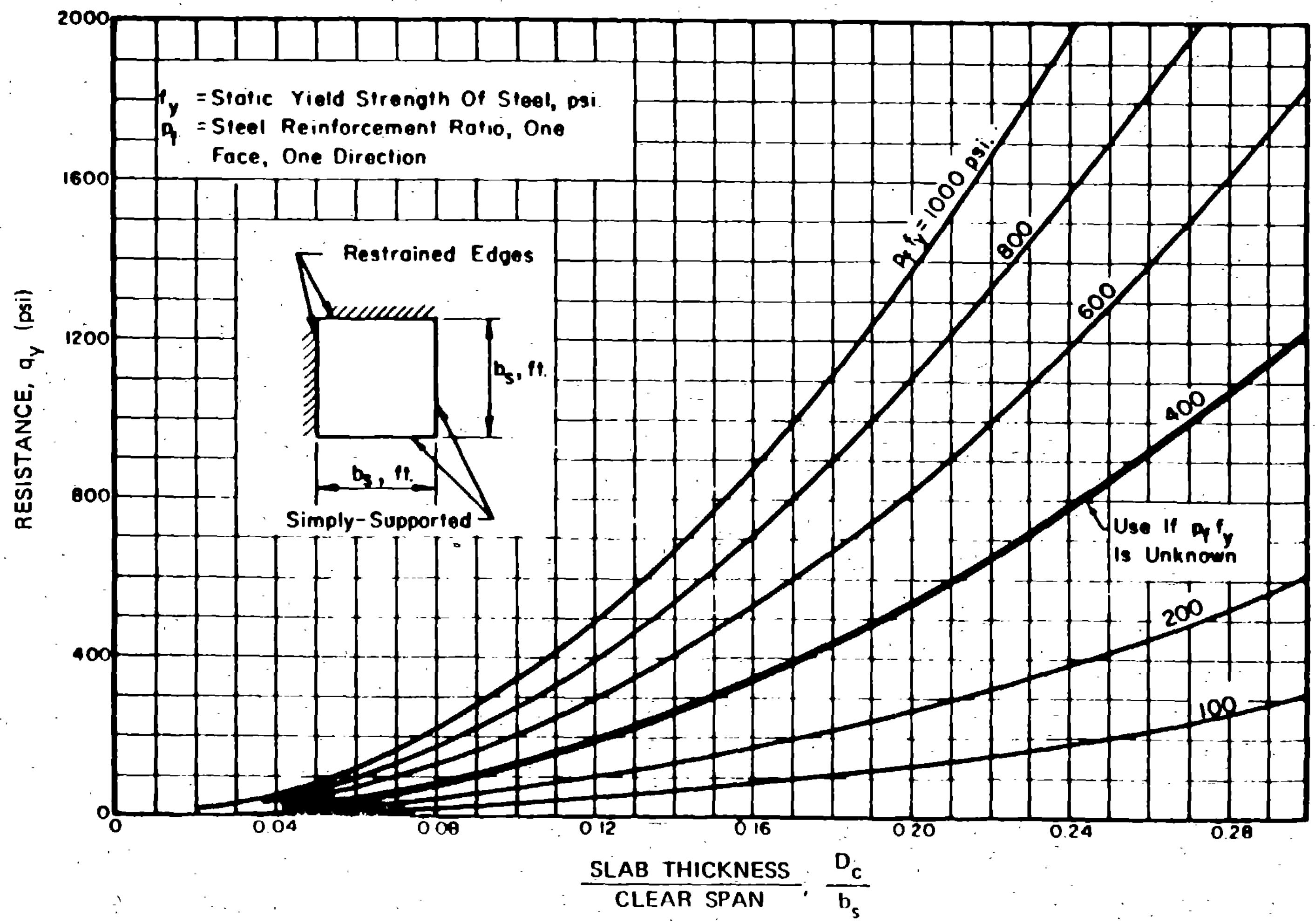


Figure 11-40. [redacted] Resistance of Horizontal Square Two-Way Slabs – Two Adjacent Edges Supported [redacted]

[redacted]

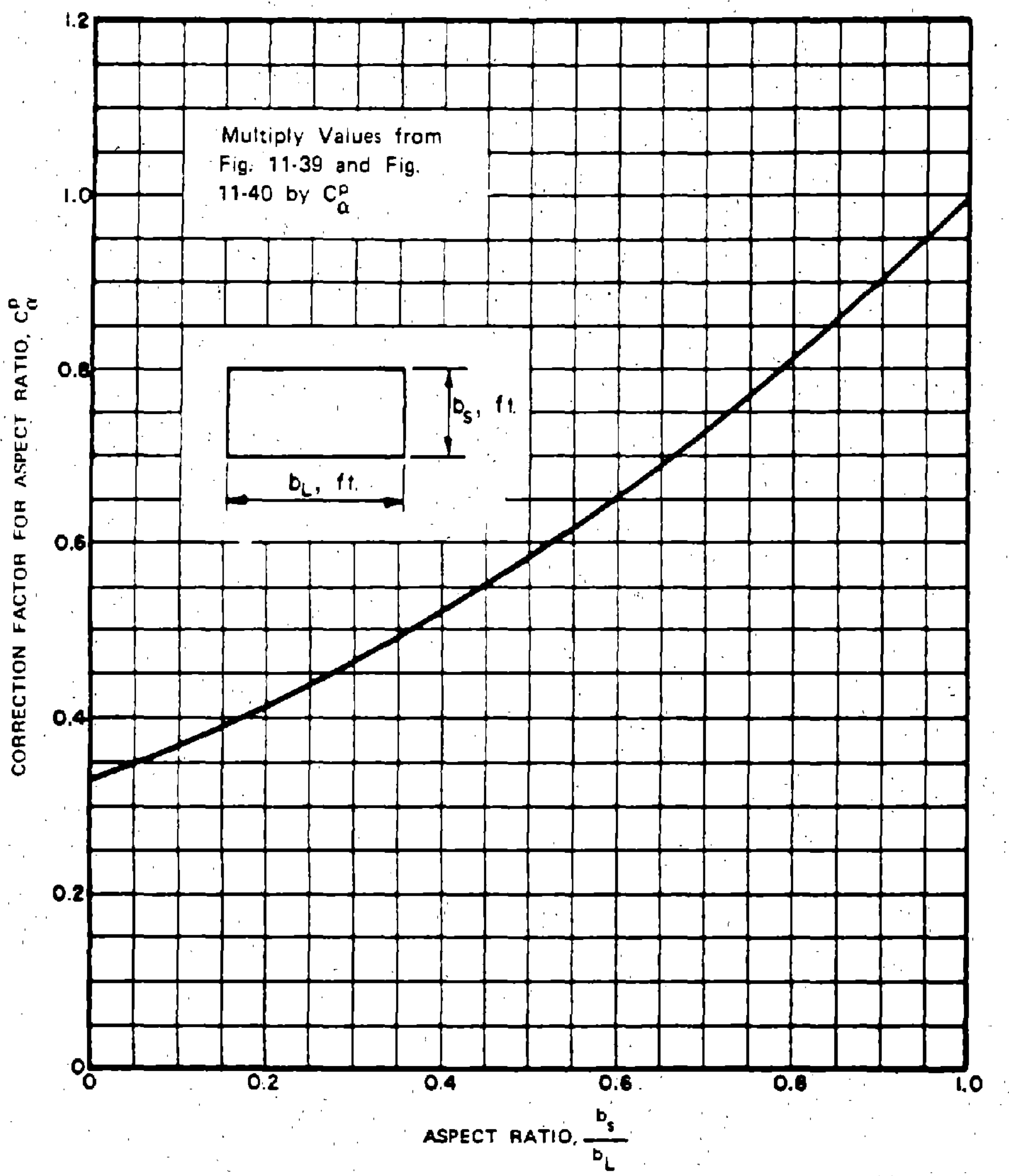


Figure 11-41. [redacted] Aspect Ratio Correction Factor for Strength of Two-Way Slabs [redacted]

[redacted]

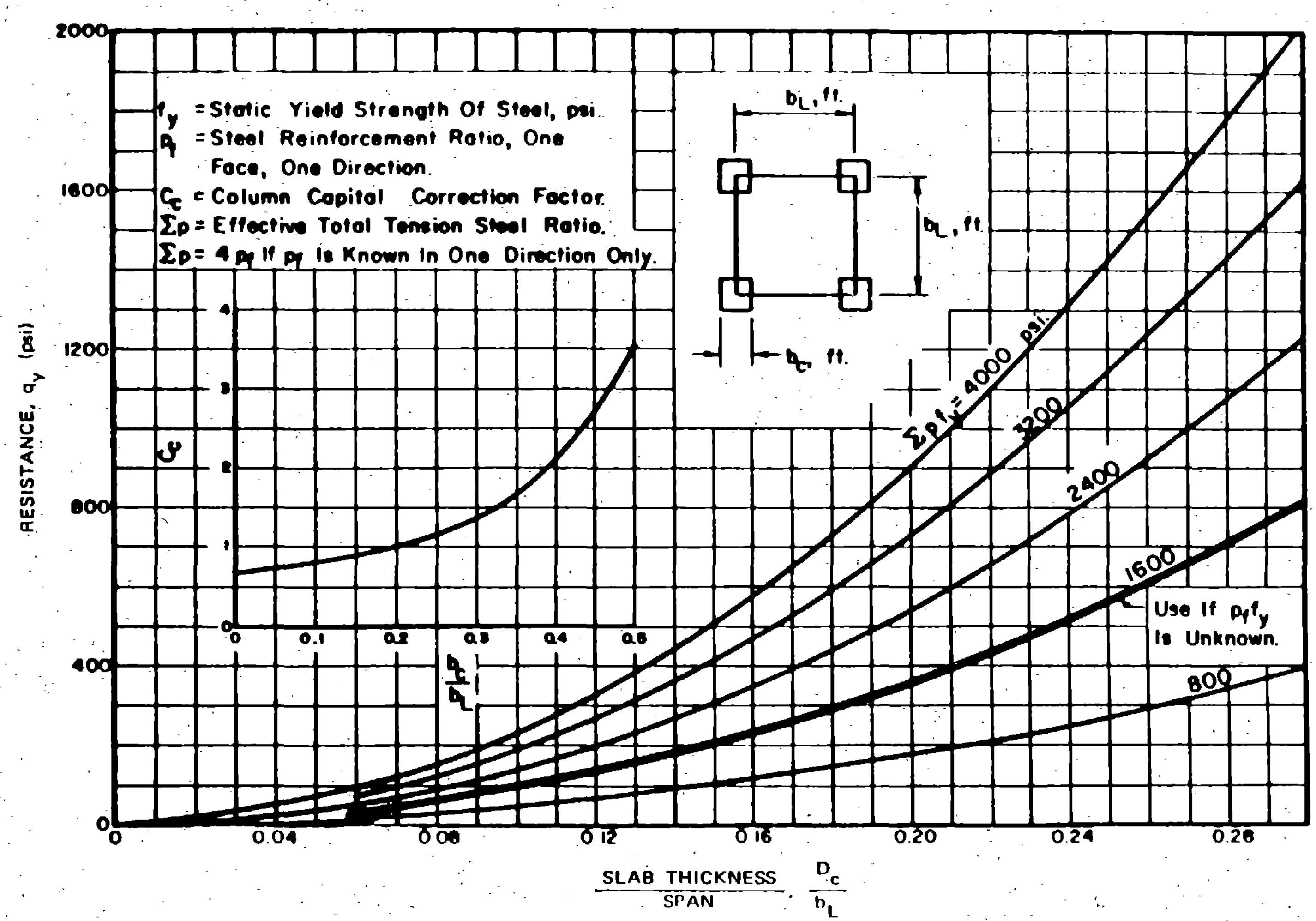


Figure 11-42. [redacted] Resistance of Horizontal Square Flat Slabs [redacted]

[redacted]

[redacted]

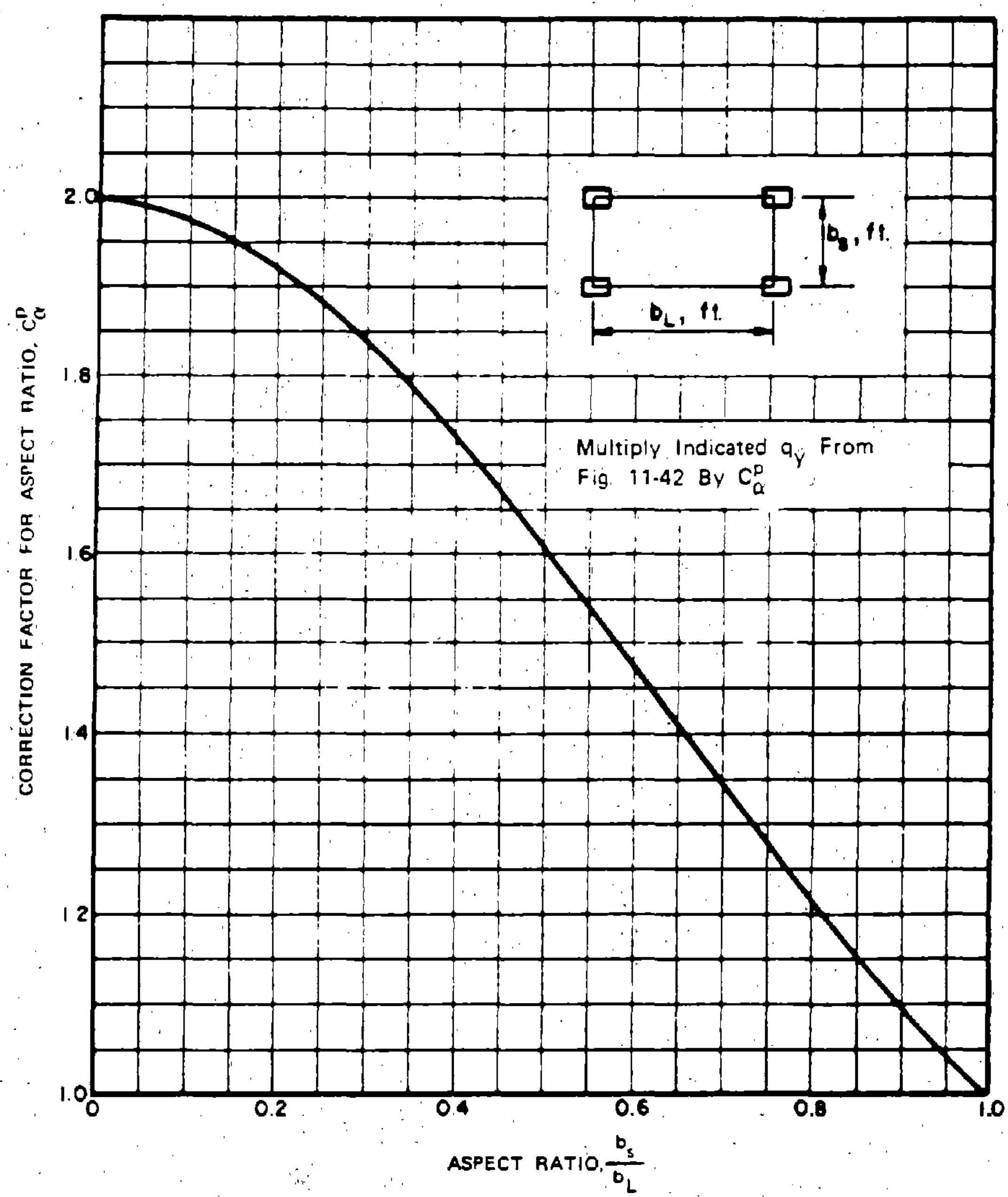


Figure 11-42a. [redacted] Aspect Ratio Correction Factor for Strength of Rectangular Flat Slabs [redacted]

[redacted]

[redacted]

NOTE: For Soils below the Water Table,

1. Determine p_H from this Chart
2. Take Half of the Value Found in (1)
3. Add 0.4z, where z is in Feet, to the Value Found in (2) to Get p_H for Use in Subsequent Calculations

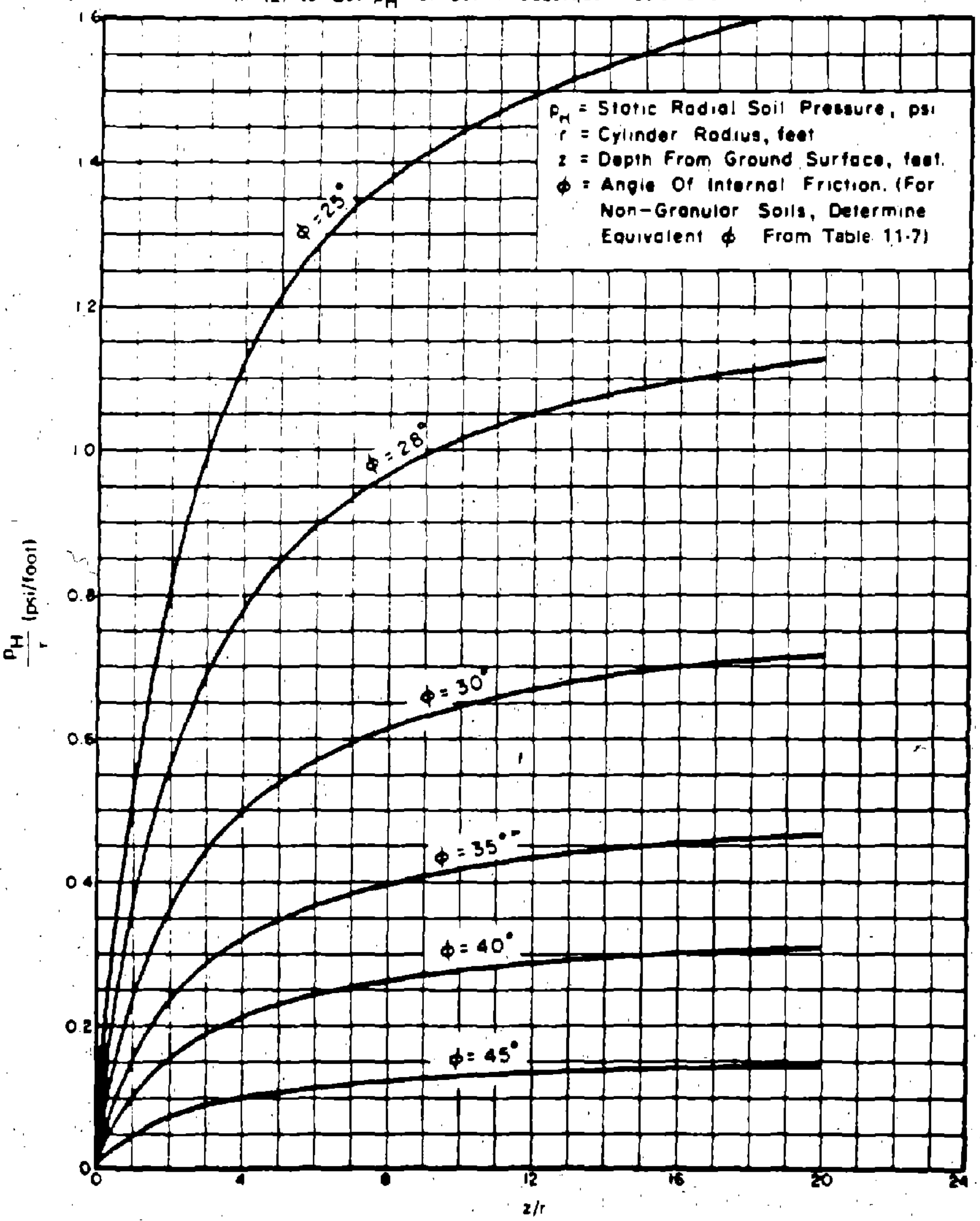


Figure 11-43. [redacted] Radial Dead Load Soil Pressure on Vertical Cylinder [redacted]

[redacted]

[redacted]

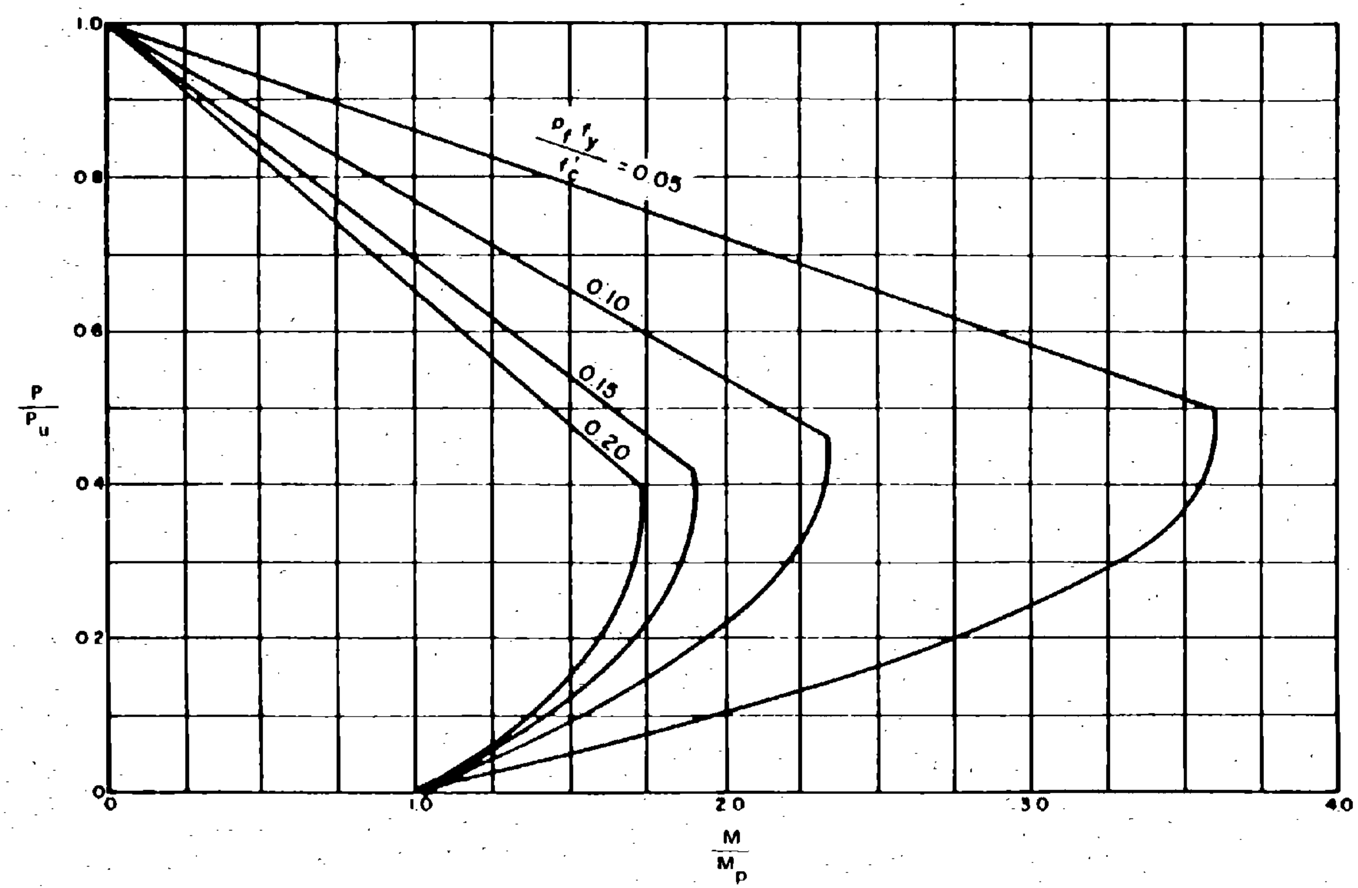


Figure 11-44. [redacted] Interaction Diagram for Reinforced Concrete Beam – Columns [redacted]

[redacted]

Problem 11-7 Calculation of the Vulnerability of Tunnels Buried in Rock

[redacted] Figure 11-45 and Table 11-8 define the vulnerability limits for tunnels in rock, with various degrees of sophistication suggested for the tunnel lining. The curves of Figure 11-45 and the slant ranges shown in Table 11-8 represent conditions that will produce a 50 percent probability of survival from a 1 megaton burst. The structure will have a significantly greater probability of survival for all greater slant ranges.

[redacted] *Scaling.* For yields other than 1 Mt, scale as follows:

For Table 11-8:

$$\frac{R}{R_1} = W^{1/3}$$

where R_1 is the slant range obtained from Table 11-8 for 1 Mt, and R is the corresponding slant range for a yield of W Mt.

For Figure 11-45:

$$\frac{R}{R_1} = 1{,}000\ (W)^{1/3}$$

where R_1 is the scaled slant range for 1 Mt, and R is the corresponding slant range for a yield of W Mt.

[redacted] *Example* [redacted]

Given: A 20 Mt weapon detonated on the surface.

Find: The slant range at which an unlined tunnel in granite will have a 50 percent probability of surviving. From Figure 11-45, the scaled range for the most competent granite is 1.6.

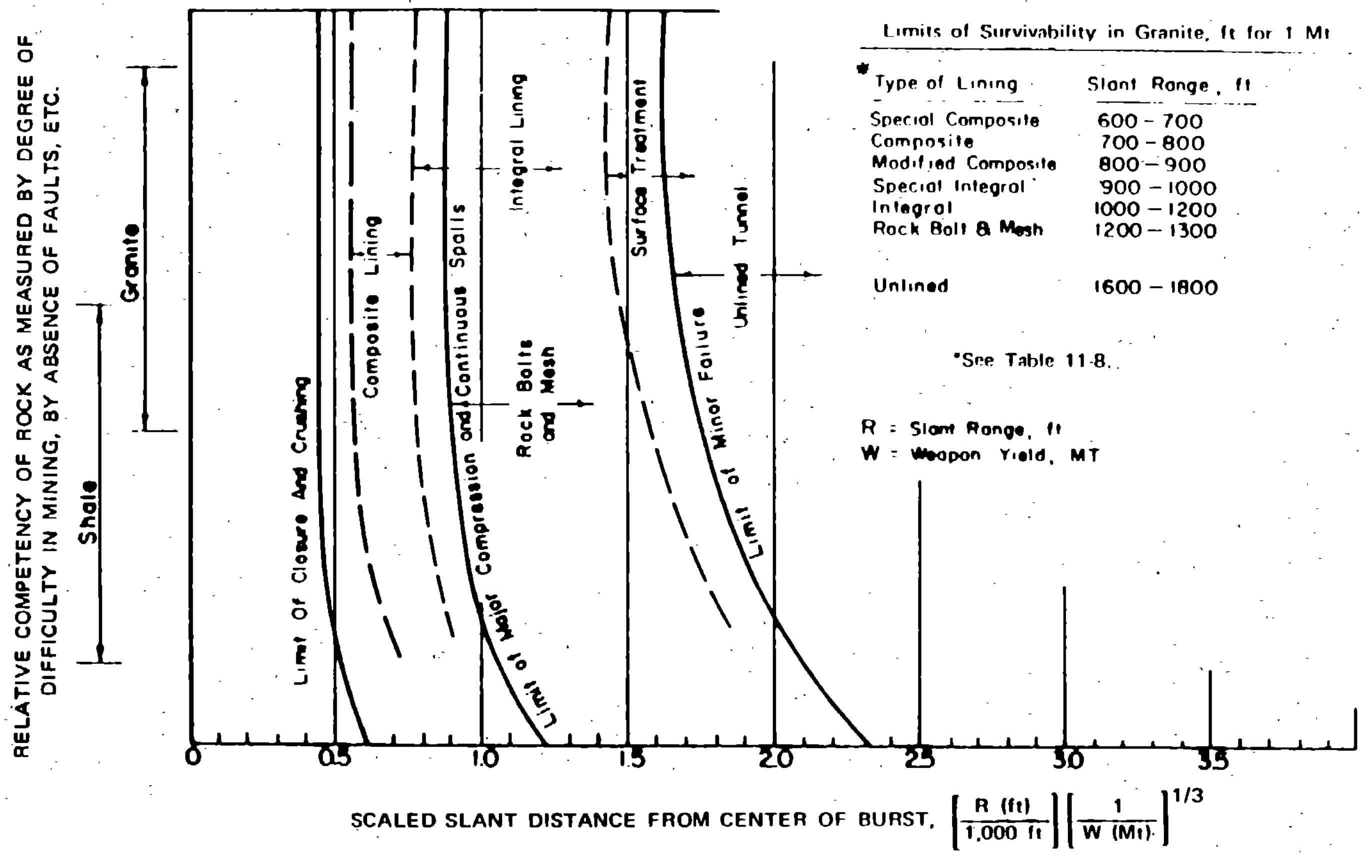
Answer: The corresponding slant range for a 20 Mt weapon is

$$R = 1{,}000 \times 1.6 \times (20)^{1/3} = 4{,}300 \text{ feet.}$$

From this result, it is obvious that some lining of the tunnel at least should be considered. For the least competent shale shown in Figure 11-45, the situation is even worse. For this medium, the scaled range shown in Figure 11-45 is 2.1. The corresponding slant range for a 20 Mt weapon is

$$R = 1{,}000 \times 2.1 \times (20)^{1/3} = 5{,}700 \text{ feet.}$$

[redacted]



Limits of Survivability in Granite, ft for 1 Mt

*Type of Lining	Slant Range, ft
Special Composite	600 – 700
Composite	700 – 800
Modified Composite	800 – 900
Special Integral	900 – 1000
Integral	1000 – 1200
Rock Bolt & Mesh	1200 – 1300
Unlined	1600 – 1800

*See Table 11-8.

Figure 11-45. [redacted] Basic Chart for Vulnerability of Tunnels in Rock [redacted]

[redacted]

[redacted]

SECTION III

[redacted] SHOCK VULNERABILITY OF EQUIPMENT AND PERSONNEL [redacted]

[redacted] As described in Section III. Chapter 2, the ground motions from the explosion of a nuclear weapon may be transmitted through the earth in a variety of ways. This section is concerned with the problem of attachment of equipment (mechanical, electrical, hydraulic, etc.) to the protective structure, or, alternatively, the vulnerability of such equipment. The question of the vulnerability of personnel is more difficult. One possible representation of personnel vulnerability is presented.

[redacted] This section is closely related to the material contained in Section III, Chapter 2, and no attempt should be made to use the material in this section for anything other than a qualitative understanding of the problems associated with shock vulnerability of equipment and personnel without first having an understanding of Section III, Chapter 2.

11-15 Shock Mounting [redacted]

[redacted] The problem of vulnerability of equipment can be related directly to the attachment of the equipment to the protective structure. The equipment must remain attached throughout the duration of the shock and must function in the postburst state. It is obvious that the attachments must have sufficient strength to transmit the forces that are associated with the equipment accelerations and with the relative distortions of structure and equipment. The stiffness of the attachments must be considered not only in relation to its influence on the magnitudes of transmitted forces but also in relation to possible limits of acceptable relative displacements and accelerations of the equipment and the structure.

[redacted] Since the problem relates to the mounting of equipment, rather than to the design of major structural components, it can be assumed that the attached mass is relatively small compared to the mass of the structure. It follows that the attachment forces are negligible in comparison with the direct effects of the explosion, and the motion of the structure is nearly independent of the forces transmitted through the attachments. Motion of the structure is assumed to be the basic input for which the mounting must be designed. These input data must be obtained from an analysis of the response of the structure to ground shock and air blast, as described in Section II of this chapter and in Section III, Chapter 2.

[redacted] Maximum accelerations or displacements that can be tolerated by the equipment must be known or must be computed. For complex items, such as electronic equipment, this information may be supplied by the manufacturer. Typical vulnerability data for various types of equipment are tabulated in Table 11-11. The vulnerability is not simply defined by single frequency and acceleration limits, e.g., there generally is a range of acceleration and frequency over which the item (or class of items) may be vulnerable. For example, for heavy equipment the range might be that shown in Figure 11-46 for the shock mounted and nonshock mounted cases. The values shown in Table 11-11 represent the midpoints of these zones. These values are used in the vulnerability computations described later.

[redacted] As mentioned previously the determination of the vulnerability of personnel is even more difficult. One possible representation for personnel is sketched in Figure 11-47, wherein it is seen to be a line representation.*

* [redacted] A more detailed discussion of vulnerability data and analysis may be found in *Vulnerability Handbook for Hardened Installations*, 1965 (See Bibliography).

[redacted]

[redacted]

Table 11-11. [redacted] Estimates of Frequency and Vulnerability of Typical Items of Equipment [redacted]*

Class	Item	Shock Mounted	Typical Value of: Fundamental Natural Frequency (cps)	Typical Value of: Estimated Vulnerability-Level Acceleration (g)
A	Heavy machinery—motors, generators, transformers, etc. (> 4,000 lb)	No Yes	10 3	20 40
B	Medium and light—pumps, condensers, air conditioning, fans, small motors (< 1,000 lb)	No Yes	20 5	40 80
C	Communication equipment, relays, rotating magnetic drum units of electronic equipment, etc.	No Yes	25 6	7 60
D	Storage batteries, piping and duct work	No Yes	20 5	70 150
E	Personnel	(see text)	–	–

*[redacted] Fifty percent probability of severe damage.

[redacted]

[redacted]

11-16 Nature of Elastic Systems Comprised of Mounted Equipment [redacted]

[redacted] In general, any piece of mounted equipment comprises a multidegree-of-freedom elastic system (or elasto-plastic system) that responds to the motion of its support points (points of attachment to the structure). If the equipment is connected to the structure in a manner such that relative distortions of the structure can be accommodated without serious stresses in the equipment and attachments, the stresses in the equipment and forces transmitted through the attachments will be primarily a function of accelerations of the equipment. Thus, the major problem of analysis is the determination of equipment accelerations. The products of equipment masses (concentrated or distributed) and corresponding accelerations represent a loading for which the corresponding stresses and support forces can be found by conventional methods of stress analysis.

[redacted] Every system has many degrees of freedom and corresponding modes of motion, and the total motion is comprised of the sum of the response in each mode. Fortunately, most systems have only a very few, easily recognized modes of predominant significance, which contribute most of the response to a specified direction of support motion. Consequently, it usually is sufficient to determine the response in each (often only one) of these predominant modes. When it is necessary to determine the response in more than one mode, the fact that peak values of stresses and reactions in the separate modes are unlikely to occur simultaneously should be considered in order to simplify the analysis. The combination of values from the separate modes should be based on probability considerations.

[redacted] In some instances, the flexibility of a piece of equipment and its attachments may be limited almost entirely to that of the attachments. For example, this is the case if an electric motor is attached to the structure by relatively soft spring mountings. In other cases, the attachments may be very rigid and the equipment may be relatively flexible. An example of the latter would be piping having a relatively small ratio of diameter to distance between points of support.

[redacted] In many instances for which the equipment has a mass distributed over considerable length, or area, it is convenient to approximate the distributed mass by one (or a few) mass concentrations.

11-17 Design of Mounted Equipment to Resist Shock [redacted]

[redacted] In a typical case, an underground structure may be considered to move with the ground in accordance with the free-field motions at or near the base of the structure. If a piece of equipment is to be mounted in the structure, the equipment must be designed for the response it would receive. This response is determined by the frequency of the system composed of the piece of equipment, its mounting bracket or connections, and the part of the structure to which it is attached. In general, the structure will be sufficiently rigid that all parts of the structure will have the same motions. Consequently the input motion for which the equipment is to be designed is the free-field earth motion.

[redacted] If the equipment is a heavy, compact element mounted on a bracket, an estimate of the natural frequency of the system must be made. In most cases it will be possible to assume that the point of attachment of the bracket to the wall of the structure is a fixed point of support. Then, from the flexibility of the bracket and the magnitude of the supported mass, the natural frequency can be calculated. This can be estimated fairly well by determining what the deflection of the system would be in the direction of motion as a result of a force equal to the

[redacted]

[redacted]

weight of the supported element. If this deflection is x_s, then the frequency f is approximately

$$f = \frac{1}{2\pi}\sqrt{\frac{g}{x_s}}$$

where g is the acceleration of gravity.

[redacted] In general, it is desirable to provide as much flexibility in the mounting as possible without sacrificing strength. This will make the response of both the equipment and the mounting as slow as possible.

[redacted] The shock spectrum may be used in very nearly the same way to design for a limiting condition of acceleration, relative velocity, or displacement relative to ground.

11-18 Vulnerability Analysis [redacted]

[redacted] If the vulnerability criteria, in terms of frequency, and acceleration, velocity, or displacement are known, a vulnerability analysis may be carried out. The simplest procedure involves choosing the vulnerability coordinate, for example, that represented by the large dot in Figure 11-46 for nonshock mounted equipment of Type A (see Table 11-11). Then, by appropriate trial and error, for a given value of weapon yield and overpressure, or range, for a given medium (c_p value), depth, etc., a spectrum can be constructed. The range or overpressure can be changed (if these are the variables, as is common) until the spectrum just encloses the point as demonstrated in Figure 11-46. The spectrum required for this analysis should correspond to the input motions at the base of the equipment support. For items of equipment mounted on exterior walls, floors, or roofs, the base motion is usually assumed to be the same as the free-field motions in the adjacent soil. For items of equipment mounted on interior structural elements, the base input motions must be determined to reflect the motion of the structural elements on which the equipment is mounted.*

[redacted] Such calculations were carried out for Equipment Classes A, B, and C (Table 11-11) for average effective seismic velocities of 1,500 fps, 5,000 fps, and 15,000 fps for 0.1, 1, and 10 Mt weapons; at depths ranging from the surface to 300 feet. It was assumed in these computations that the applicable spectra were the free-field spectra, i.e., not altered by structural behavior. The results are shown in Figures 11-48, 11-49, and 11-50.

[redacted] For the soft medium shown in Figure 11-48, c_p = 1,500 fps, the response is largely governed by air-induced motion. For the firmer medium, c_p = 5,000 fps, the response is controlled by a mixture of both air-induced and direct-transmitted motions, and for the stiff medium, c_p = 15,000 fps, the response is controlled solely by direct-transmitted motions.

*[redacted] Detailed methods for handling such cases are described in Newmark, *Notes on Shock Isolation Concepts*, 1966 (See Bibliography).

[redacted]

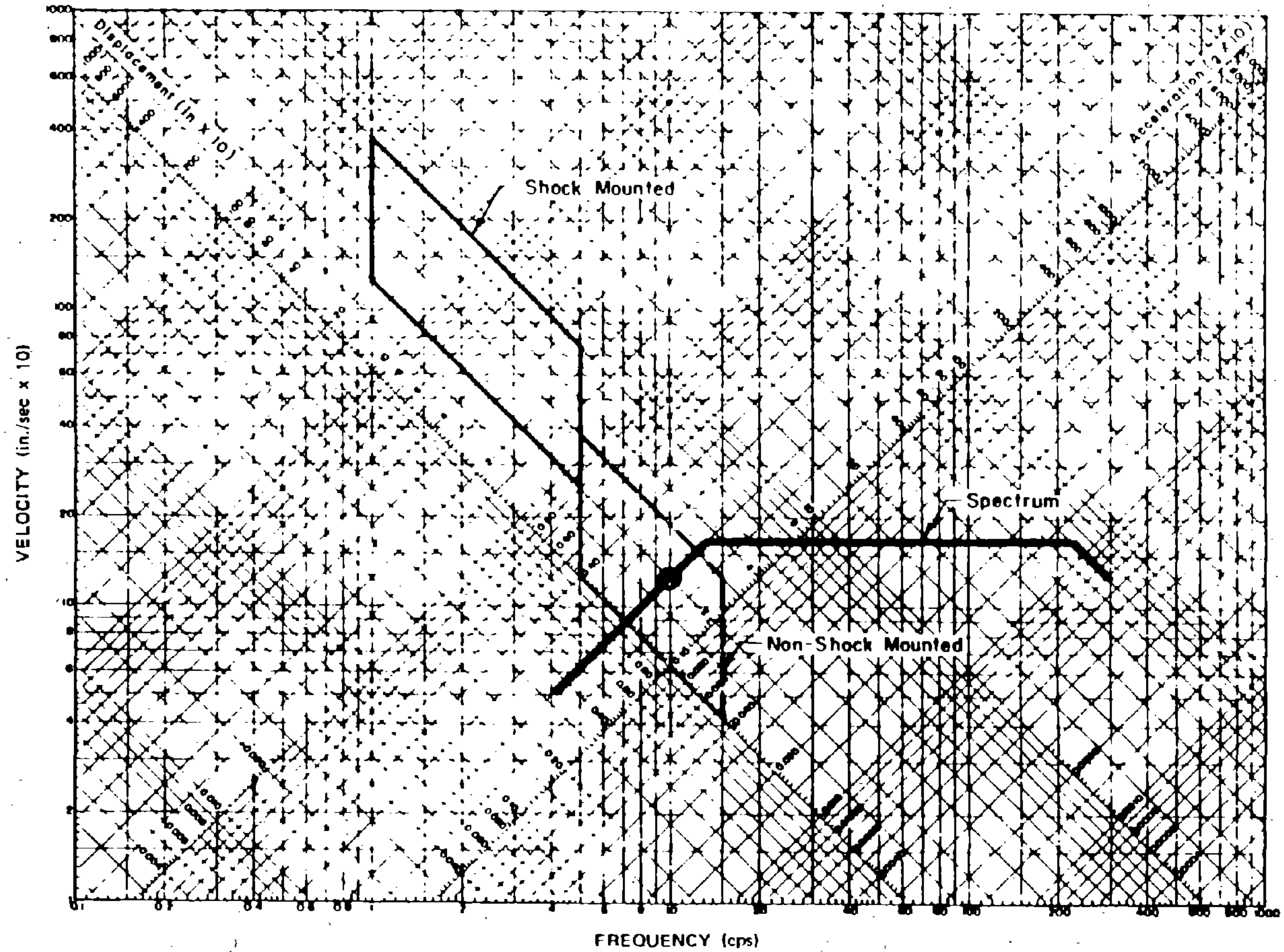


Figure 11-46. [redacted] Vulnerability Criteria – Class A Equipment [redacted]

[redacted]

[redacted]

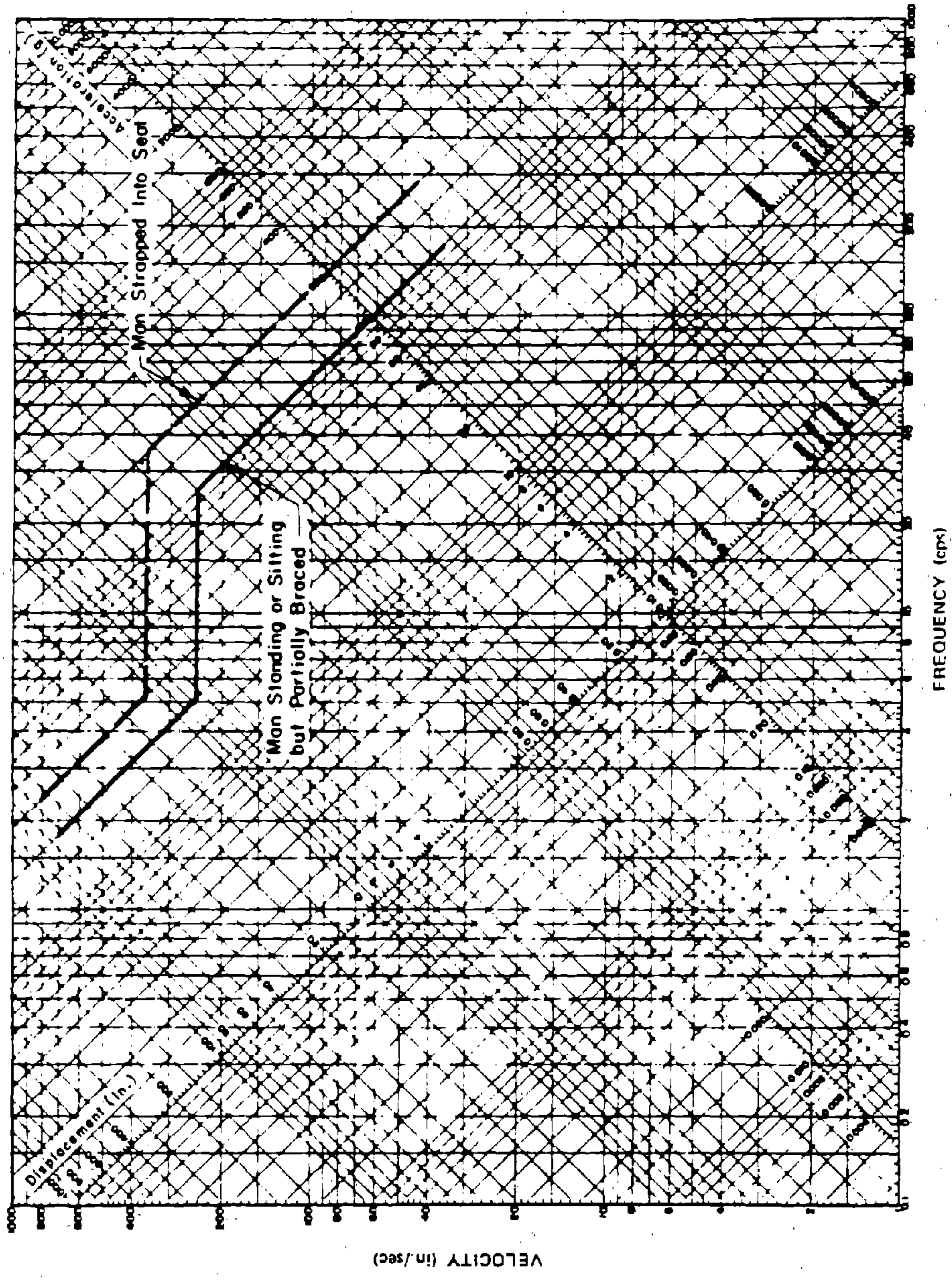


Figure 11-47. [redacted] Vulnerability Criteria for Personnel [redacted]

[redacted]

[redacted]

Problem 11-8 Calculation of the Vulnerability of Equipment Mounted in an Underground Structure

[redacted] Figures 11-48 through 11-50 show the vulnerability plots for equipment classes A, B, and C (Table 11-11) for average seismic velocities of 1,500, 5,000, and 15,000 fps, respectively. The calculations for Figures 11-48 through 11-51 were performed by the methods for determining shock spectra that are described in Section III, Chapter 2. The procedure for making a vulnerability analysis for shock effects involves the shock response spectrum and is one of trial and error. The vulnerability plots of Figures 11-48 through 11-51 simplify such analyses.

[redacted] *Scaling.* The curves of Figure 11-48 through 11-50 cannot be scaled to other yields; however, interpolation between the curves will provide an estimate as to whether a detailed shock response spectrum analysis is required.

[redacted] *Example* [redacted]

Given: An underground structure has its bottom floor located 50 feet below the surface of the ground in a medium that has a seismic velocity of 5,000 fps, as illustrated below. Non-shock mounted class B equipment is mounted directly on the floor.

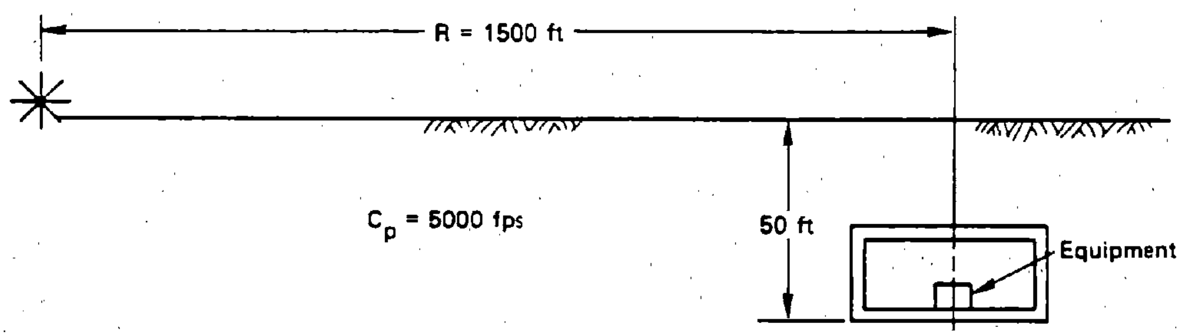
Find: The vulnerability (50 percent probability of damage) of the class B equipment if a 1 Mt weapon is burst at a ground distance of 1,500 feet from the structure.

Solution: The peak ground motions of the base of the structure must be calculated, and the corresponding spectra for vertical and horizontal motions must be sketched. The vulnerability of the equipment can then be estimated from the information contained in Table 11-11.

The burial conditions, weapon yield, and ground distance for this problem are identical to those of Problem 2-27. Therefore, the peak ground motions of the base of the structure that were obtained in Problem 2-27 apply to this case as well. These are tabulated below.

In this particular example, as was noted in Problem 2-27, the air-induced motions, with the exception of the horizontal displacement, are the largest values for both the vertical and horizontal directions. In the horizontal direction, the air-induced value of displacement is nearly equal to that of the direct-transmitted value. In many cases, especially near the burst and in a stiff medium, the direct-transmitted motions will be the largest, and will control the analysis.

As described in Section III, Chapter 2,



[redacted]

[redacted]

	Displacement. d (in.)	velocity. v (in./sec)	Acceleration. a (g)
Vertical			
Air-induced	6.3	59	59.
Direct-transmitted	0.7	15	2.4
Horizontal			
Air-induced	2.1	40	59.
Direct-transmitted	2.2	22	2.4

[redacted] the bounds for the shock response spectra are obtained by multiplying the controlling displacement, velocity, and acceleration values by 1, 1.5, and 2.0, respectively.

Vertical Response Spectrum Bounds:

Displacement d = 1.0d = 6.3 in.
Velocity v = 1.5v = 90 in./sec
Acceleration a = 2.0a = 118g

Horizontal Response Spectrum Bounds:

Displacement d = 1.0d = 2.2 in.
Velocity v = 1.5v = 60 in./sec
Acceleration a = 2.0a = 118g

These spectra bounds are shown in Figure 11-51. They constitute the shock response spectra for a single-degree-of-freedom system. It has been assumed that the motion of the base slab is the same as the free-field motion of the soil on which it rests.

Answer: From Table 11-11, the vulnerability coordinates for the nonshock mounted Class B equipment are 40g at 20 cps for both horizontal and vertical motion. This vulnerability point is shown on Figure 11-51. The horizontal and vertical spectra bounds fall substantially below the vulnerability point, indicating that a larger yield and/or shorter range will be required to produce 50 percent probability of damage to the equipment. This conclusion is substantiated by the data presented in Figure 11-49, where it will be noted that for vertical motion and Class B equipment, a range of about 1,300 feet for a weapon yield of 1 Mt would be required for 50 percent probability of severe damage. In Figures 11-48 through 11-50, the curves denote 50 percent probability of severe damage. Points to the left of a curve represent increasing degrees of damage from that weapon yield. Points to the right of a curve represent less than 50 percent probability of damage.

Related Material: See paragraph 11-18. See also Section III, Chapter 2 and Problem 2-27.

[redacted]

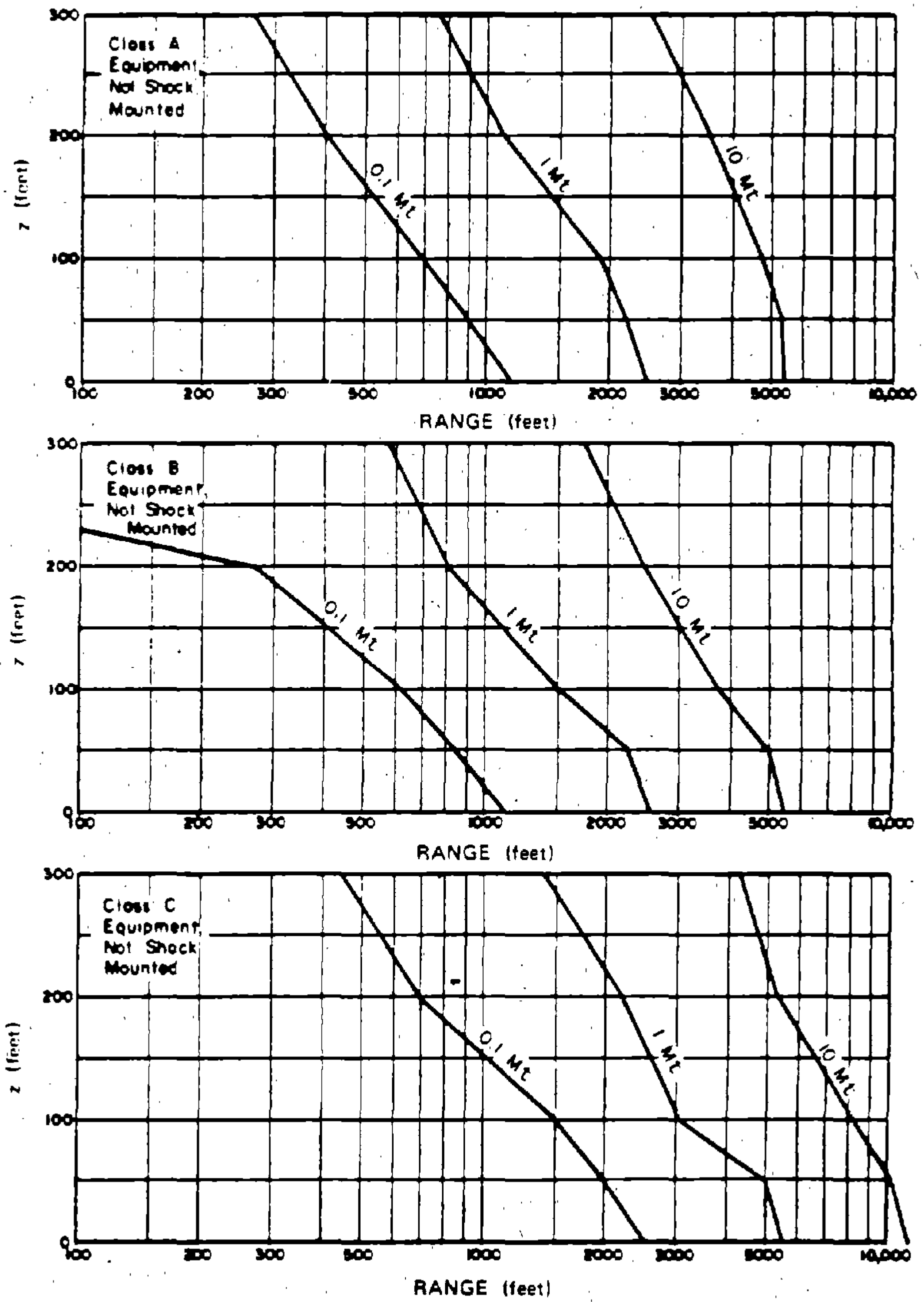


Figure 11-48. [redacted] Vulnerability Plot for Seismic Velocity (c_p) of 1500 fps — Vertical Motion [redacted]

[redacted]

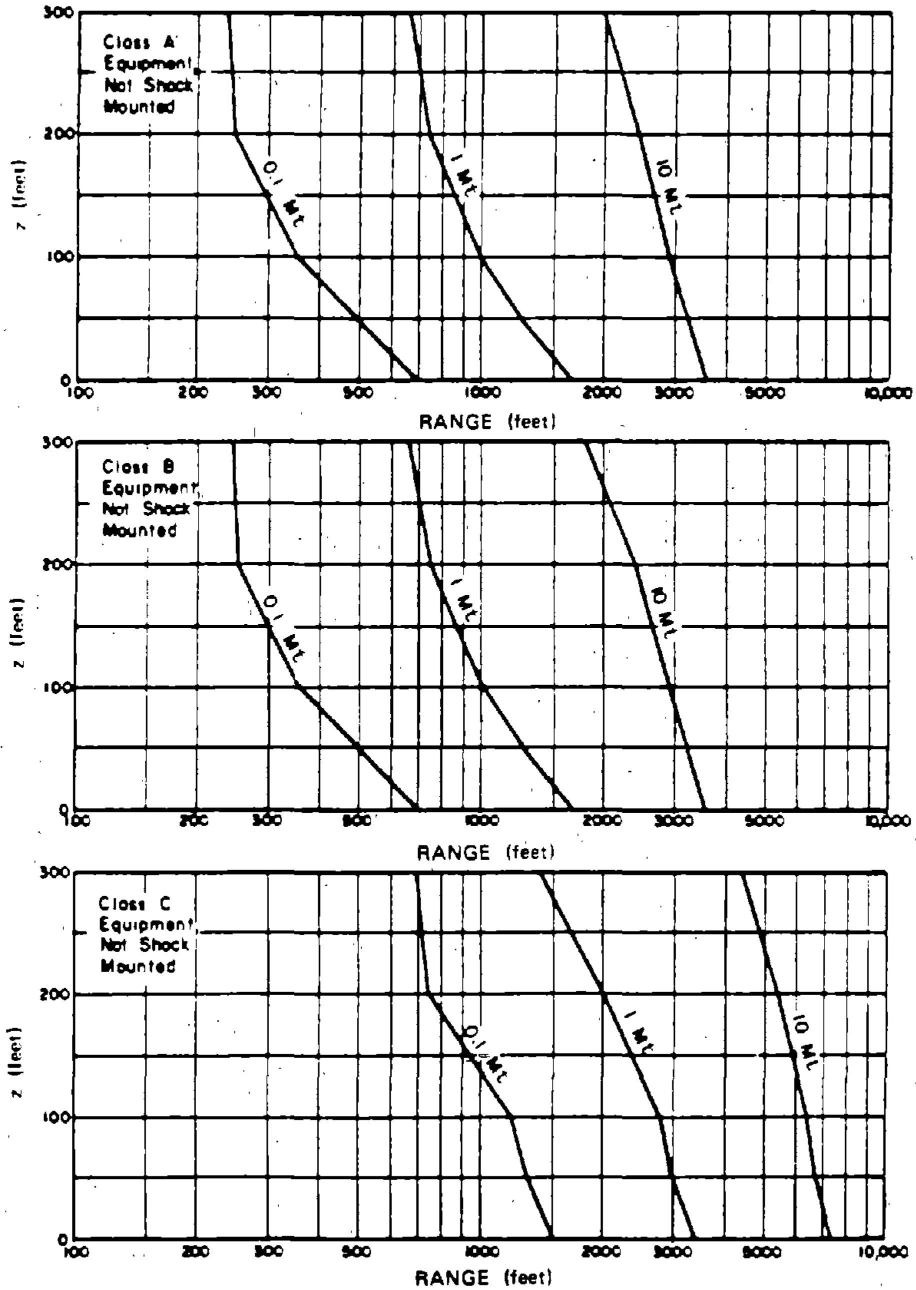


Figure 11-49. [redacted] Vulnerability Plot for Seismic Velocity (c_p) of 5000 fps – Vertical Motion [redacted]

[redacted]

[redacted]

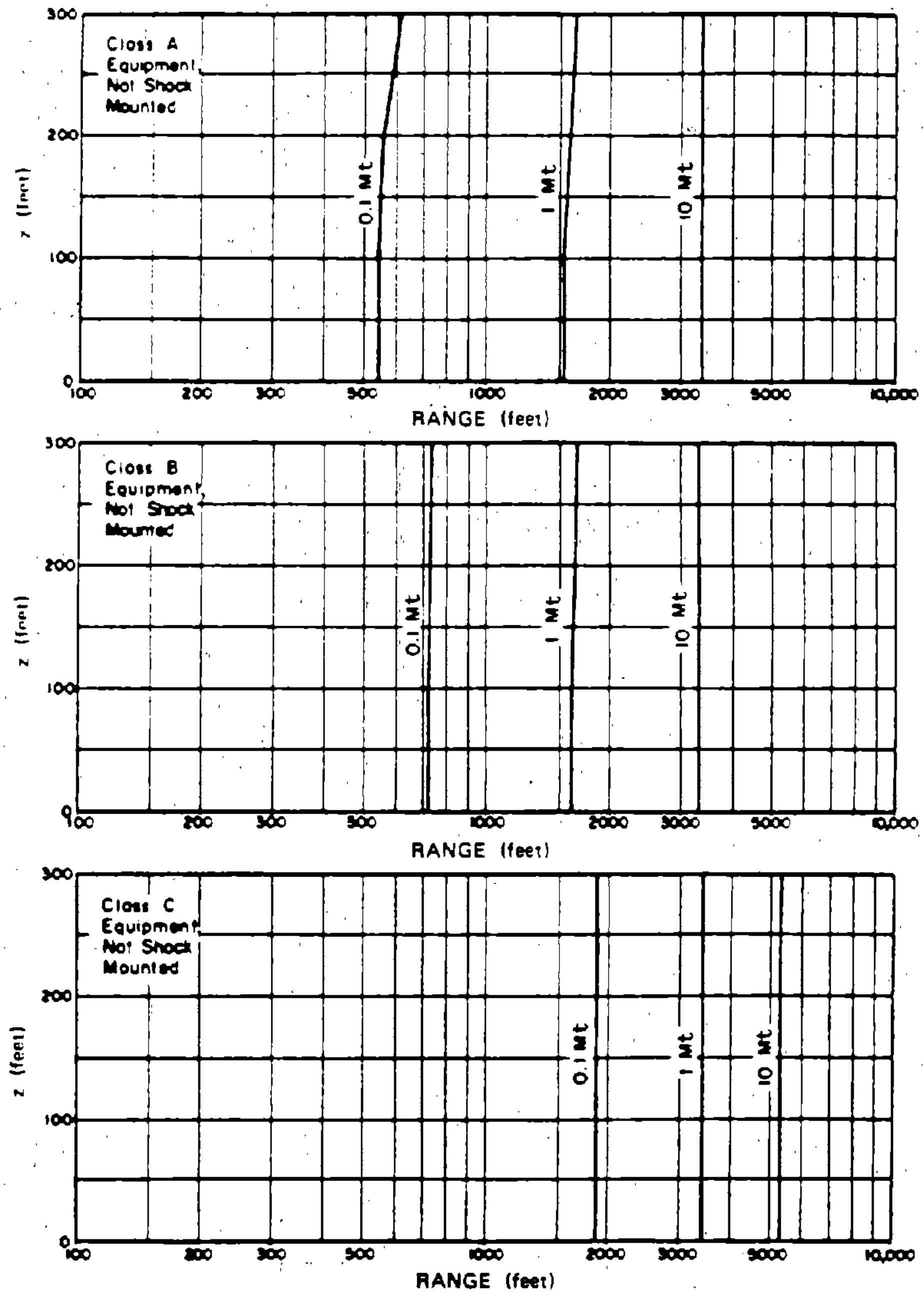


Figure 11-50. [redacted] Vulnerability Plot for Seismic Velocity (c_p) of 15,000 fps – Vertical Motion [redacted]

[redacted]

[redacted]

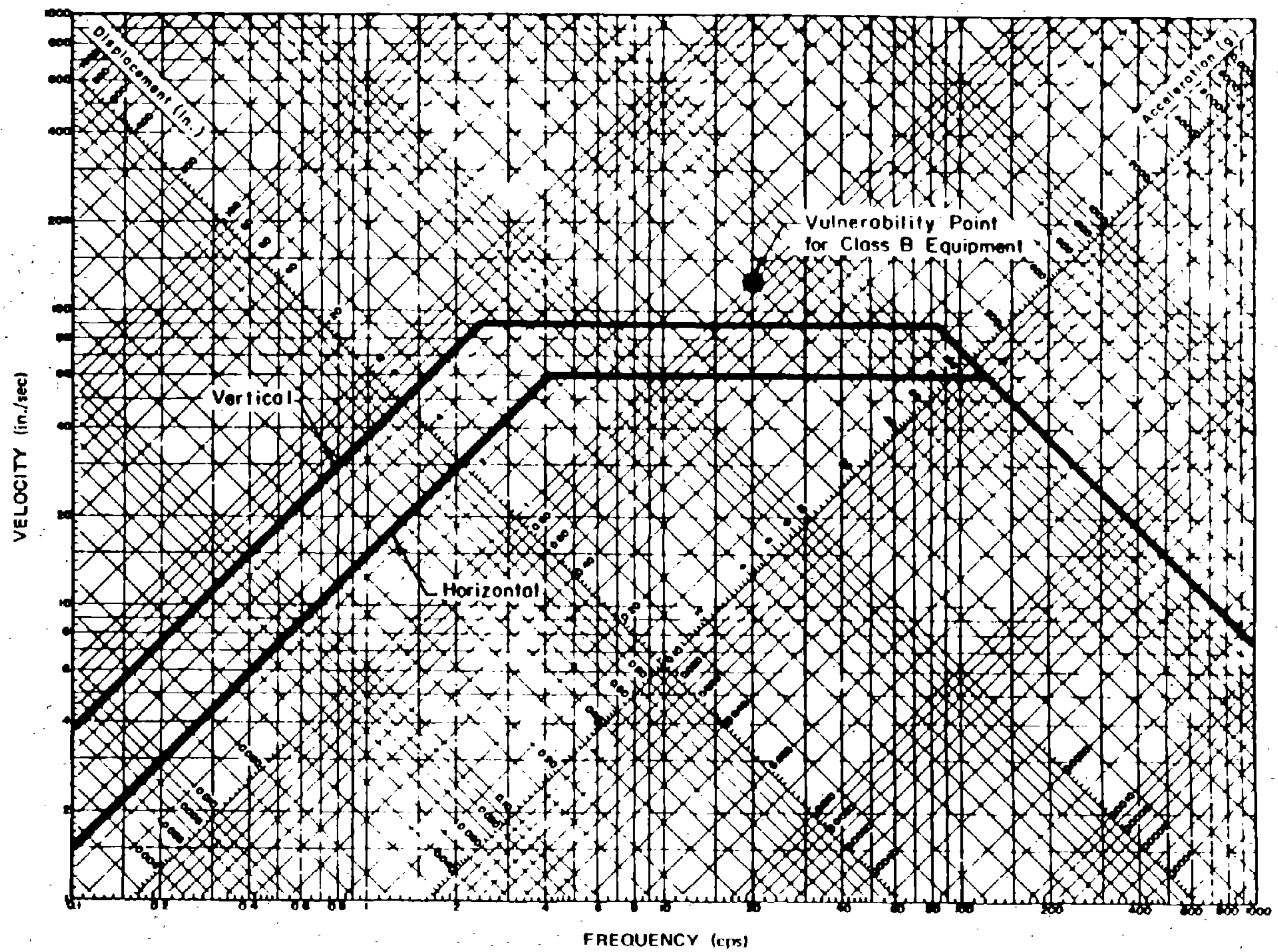


Figure 11-51. [redacted] Shock Response Spectra and Vulnerability Calculation [redacted]

[redacted]

SECTION IV
[redacted] DAMS AND HARBOR INSTALLATIONS (U)

[redacted] AIR BLAST [redacted]

11-19 Concrete Gravity Dams [redacted]

[redacted] A concrete gravity dam with reservoir water depth less than about half the dam height is most vulnerable to an upstream surface burst and, conversely, least vulnerable from a downstream burst. A downstream or overhead burst is, however, a primary damaging mechanism for powerhouse structures; these should be analyzed according to structural type as in paragraph 11-4.

11-20 Harbor Installations [redacted]

[redacted] Air blast is the most important damaging mechanism for most structures around a harbor (see the discussion in paragraphs 11-1 through 11-4). Air blast can make the canal or river locks, where the water level around the gates is low, inoperable by damaging the gates.

[redacted] WATER SHOCK [redacted]

11-21 Concrete Dams and Water Locks [redacted]

[redacted] A concrete gravity dam with reservoir water depth higher than about half the dam height is most vulnerable to an underwater burst. Vulnerability increases with depth of burst since underwater shock impulse for a given yield at a given range is greater for greater depths. Only a limited amount of information is available concerning dam destruction, and scaling laws are not well known. The following slant ranges are estimated for damage by a 20 kt underwater burst at mid-depth to full concrete gravity dams (straight or slightly curved in plan):

- 60-foot high dam: Cracks are produced at a range of about 1,000 feet; portions are cracked loose and displaced small distances at a range of about 600 feet.
- 150-foot high dam: Cracks are produced at a range of about 1,500 feet; portions are cracked loose and displaced sizable distances at a range of about 600 feet.
- 500-foot high dam: Cracks are produced at a range of about 4,000 feet; portions are cracked loose and displaced large distances at a range of about 600 feet.

[redacted] Canal and river locks, where there is a high water level around the gates, are most vulnerable to damage from an underwater burst.

[redacted] CRATERING [redacted]

11-22 Cratering Earth Dams and Causeways [redacted]

[redacted] The primary damage mechanism for earth dams and causeways is cratering. The dam or causeway should be within the crater in order to insure breeching the structure.

[redacted] Although the crater lip formed by an underwater burst in a harbor may create a navigational hazard, water erosion may make the hazard temporary. The range of air-blast damage to harborside structures is greater than that of cratering damage from an underwater burst near the shore. Cratering is the most important damage mechanism for concrete quaywalls and canal and river locks, if the structure is within the rupture zone. The dimensions of the crater formed by an underwater burst can be computed using the procedure given in Problem 2-36. Dimensions of craters from ground surface and underground bursts can be computed from the procedures given in paragraph 2-48. Weapons that explode on the top or at the toe of a concrete gravity dam will produce damage to the dam by cratering. The extent of the rupture can be calculated by the method given in paragraph 2-46 through 2-50. The extent of rupture resulting

[redacted]

from a burst on top of the dam determines the amount that the water level will drop. For a burst at the toe of the dam, the extent of rupture also will determine whether the dam will overturn. The extent of damage resulting from an explosion in the dam gallery can be calculated in the same manner as described for an underground burst by the method given in paragraph 2-50.

[redacted] WATER WAVES [redacted]

[redacted] The many variables involved in predicting damage from wave action require an individual analysis of each target structure. Among the variables are water depth, bottom slope, wave height, wavelength, target response characteristics, orientation of target to wavefront, location of target relative to the point of wave breaking, and variation in width of the channel or harbour. Section IV, Chapter 2 provides estimates of the maximum wave height as a function of water depth and burst position. These values are given for a constant depth of water. The wave height changes as the water depth or width of the wavefront varies. Wave action may cause additional damage to structures that already have been damaged by air blast. Wave damage may result from impact and hydrostatic pressure, drag force, or inundation.

11-23 Impact and Hydrostatic Pressure [redacted]

[redacted] The magnitude of the impact force depends upon the velocity and mass of the wave and upon whether the wave has broken or is breaking at the time of impact. The hydrostatic pressure depends upon the wave height beside the structure. Since the lower limiting velocity for damage to light structures by impact is obtained by very-low-amplitude waves, and since the velocity increases with amplitude, a wave with a height sufficient to reach inland structures must be considered a probable damaging agent.

11-24 Drag Forces [redacted]

[redacted] Objects around which a wave may easily pass such as piling, are subject to drag forces. A wave passing by a ship may cause displacement as a result of drag forces, and damage may occur as a result of the ship colliding with some other object, such as a pier.

11-25 Inundation [redacted]

[redacted] When a long-lasting wave from a nuclear explosion reaches a sloping beach, it increases in height and run inland. Large areas may be temporarily inundated.

[redacted] THERMAL-RADIATION DAMAGE [redacted]

[redacted] Dams, whether of earth or concrete, are not significantly affected by thermal radiation. However, since many waterfront areas contain a high concentration of combustibles, large fires can result if these are ignited.

SECTION V

[redacted] PETROLEUM, OIL, AND LUBRICANT (POL) STORAGE TANKS [redacted]

11-26 Damage Criteria [redacted]

[redacted] Although petroleum, oil, and lubricant storage tanks are structures, their behavior under blast loading is very complex and difficult to determine, and thus requires separate treatment. In addition to being vulnerable to blast effects, these tanks are susceptible to primary and secondary fire as well as to loss of contents from penetration by random fragments and missiles in the blast environment. However, since their blast vulnerability is likely to be either as great or greater than their vulnerability to missiles, except at high overpressures, information developed to date that is reflected in the damage curves of Figures 11-52 through 11-55 applies only to blast effects. The curves are based on the limited

[redacted]

[redacted] data derived from nuclear field tests, shock-tube experiments, and analytical studies.

11-27 Loading and Response [redacted]

[redacted] Tests indicate that standard storage tanks that are designed under American Petroleum Institute specifications will fail through the peeling up or "uplift" of the shell plates facing ground zero at or near the junction with the bottom plate. Many parameters affect both the loading and the response of a tank; those reflected in the damage curves are yield, diameter of tank, height-to-diameter ratio, degree of filling, and type of tank. Typical values were chosen. Damage levels are about the same for floating-roof and cone-roof tanks. Since a cone roof is very weak, it probably will be blown off quickly, and it is unlikely that a floating roof will restrain the motion of the tank shell for any length of time at loadings sufficient to cause failure. The wind girder of a floating roof has little effect on response. Concrete-protected tanks of either type are more stable, since they have reinforced-concrete rings surrounding their steel sides. For such tanks, standard service drawings for diameters of 134, 120, 67, and 42.5 feet were used as references, and structural characteristics were obtained for equivalent 100-, 75-, and 50-foot protected tanks.

11-28 Damage [redacted]

[redacted] The criterion for severe damage to storage tanks is any rupture that causes the loss of the contents, or sufficient spillage to provide a high probability of fire. With respect to other structures, this criterion is related to the level of 50 percent probability of severe damage. A level of moderate damage for storage tanks is difficult to relate and is not used. Light damage, i.e., distortion and bending of the shell, roof or appurtenances, can be expected for overpressure levels given in Table 11-12.

[redacted] The overpressures determined from Figures 11-52 through 11-55 and Table 11-12 are used in conjunction with the overpressure vs height-of-burst curves for near ideal surface conditions in Section I, Chapter 2 to find damage radii for unprotected tanks. [redacted] This practice has been followed in the curves in this section.

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[redacted] The reduction in damage distances to POL tanks that will occur for shallow subsurface bursts may be obtained from Figure 11-24 by the methods described in Problem 11-2.

[redacted]

[redacted]

Table 11-12. [redacted] Light Damage to POL Tanks [redacted]

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[redacted]

[redacted]

Problem 11-9 Calculation of Damage to POL Storage Tanks

[redacted] Figures 11-52 through 11-54 contain curves that indicate the overpressures required for 50 percent probability of severe damage to various configurations of POL storage tanks filled to varying levels. Separate families of curves are provided for weapon yields of 20 kt and 500 kt. To determine the radius of damage, use the overpressure vs height-of-burst curves for near-ideal surface conditions in Section I, Chapter 2, for unprotected tanks [redacted] To predict light damage, use the criteria of Table 11-12. Because of the great possibility that fire or explosion will cause further damage when ruptured tanks leak, no attempt has been made to define moderate damage.

TNA (2)(1)

[redacted] *Scaling:* Use the scaling that accompanies the appropriate overpressure vs height-of-burst curve.

[redacted] *Example 1* [redacted]

Given: An unprotected conical roof tank 100 feet in diameter, 35 feet high, and 0.5 full.

Find: The radius of severe damage from a 20 kt ground-surface burst.

[redacted]

TNA (2)(1)

[redacted]

[redacted] *Example 2* [redacted]

Given: A protected floating roof tank 50 feet in diameter, 30 feet high, 0.9 full, [redacted]

Find: The radius of severe damage when subjected to the blast from a 1 Mt weapon detonated 3,000 feet above the ground surface.

[redacted]

Related Material: See paragraph 11-28 and Table 11-12. See also Figures 2-17 through 2-22.

[redacted]

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Figure 11-52. [redacted] Blast Resistance of Unprotected Floating and Conical-roof Tanks for 20 kt [redacted]

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Figure 11-53. [redacted] Blast Resistance of Protected Floating and Conical-roof Tanks for 20 kt [redacted]

[redacted]

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Figure 11-54. [redacted] Blast Resistance of Unprotected Floating and Conical-roof Tanks for 500 kt [redacted]

[redacted]

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Figure 11-55. [redacted] Blast Resistance of Protected Floating and Conical-roof Tanks for 500 kt [redacted]

[redacted]

SECTION VI

[redacted] FIELD FORTIFICATIONS [redacted]

11-29 Air Blast Damage [redacted]

[redacted] Air blast is the controlling mechanism for the production of damage to field fortifications, including those that are reinforced, revetted or covered. Direct ground shock also can lead to spalling and cracking in the earth around positions. The subsequent blast wave may cause the cracked material to collapse into unrevetted trenches and foxholes, and may damage the revetments. Definitions of severe, moderate and light damage levels for various types of field fortifications are given in Table 11-13. With the exception of unrevetted trenches and foxholes, these damage levels are based on collapse and structural failure. Unrevetted trenches and foxholes have levels based on the degree of filling caused by collapse of the walls and by dust and debris. Damage to field fortifications also depends on the engineering properties of soil, i.e., soil type, cohesiveness, compaction, and moisture content. Test results show that for a given overpressure (or impulse), major differences in damage level can occur even within a single soil classification. The range to which a given damage level extends is reduced in cohesive soils. Large amounts of vegetation with thick root systems will help reinforce a relatively weak soil. Vegetation also may reduce the amount of dust that might be blown into foxholes.

[redacted] The distance at which severe, moderate, and light damage may be expected from a 1 kt burst are shown as a function of height-of-burst in Figures 11-56 and 11-57 for the revetted structures described in Table 11-13. The overpressure levels required for wall collapse of unrevetted trenches and foxholes will vary with soil properties and with weapon yield. Figure 11-58 indicates the overpressure and impulse for given yields at which severe, moderate, and light damage to unrevetted trenches and foxholes may be expected in three basic soil types. This figure is useful both to predict the stability of unrevetted foxhole walls and as an aid to determine whether revetting would be required for a particular soil type. An estimate of the response of other soil types may be made by choosing the soil type from Figure 11-58 that is most similar to the soil of interest. A partial list of soil classifications is in Table 11-14.

11-30 Protection of Fortifications from Air Blast Damage [redacted]

[redacted] Revetted emplacements resist collapse at considerably greater overpressures than unrevetted emplacements. However, the selection and use of revetting materials must be done carefully since some materials may create a hazardous condition. For example: careless use of corrugated metal, which is excellent revetting material, can entrap the occupant; interwoven sticks can become deadly spears, if the sticks should break. Light revetting materials, such as chicken wire supported burlap or pasteboard, light timber, and plywood are fairly resistant to air blast when well supported.

[redacted] Additional protection may be obtained within field fortifications by providing overhead cover. Covers tend to reduce or eliminate the hazard of multiple reflections of the blast wave, which can result in overpressures that are double or triple the incident overpressure. There are two main categories of overhead cover: flush with grade level, and above grade level. Covers flush with grade level are subject primarily to downward pressure. Flush, removable covers provide maximum protection against air blast in foxholes. Covers above grade level are subject to the additional effects of reflected pressure and

[redacted]

Table 11-13. [redacted] Damage Criteria for Field Fortifications [redacted]

Structural Type No. and Figure No.	Description of Structure	Description of Damage		
		Severe	Moderate	Light
11-57	Command post and personnel shelter, modular sections 6′ x 8′ with top 3′ to 5′ below ground surface, earth covered, and covered trench entrance	Caps and posts broken, large displacement and disarrangement of timbers, revetment failure	Some caps and posts broken moderate displacement, some revetment failure	Damage to minor components only slight displacement, occasional revetment failure
11-56	Machine-gun emplacement, 7′ x 7′, framework extends 2′ above original ground surface, has open firing ports and open trench entrance; 3′ to 5′ mound of earth covers framework and extends down to the ground surface except at openings*	Caps and posts broken, large displacement and disarrangement of timbers, revetment failure	Some caps and posts broken, moderate displacement, some revetment failure	Damage to minor components only slight displacement, occasional revetment failure
11-58	Unrevetted trenches and foxholes with or without light cover	The trench or foxhole is at least 50 percent filled with earth	The trench or foxhole is at least 10 percent but less than 50 percent filled with earth	The trench or foxhole is less than 10 percent filled with earth

*[redacted] Post, cap, and stringer construction, timber approximately 6″ x 8″, or 12″ in diameter.

[redacted]

[redacted]

Table 11-14. [redacted] Unified Soil Classification [redacted]

Major Divisions			Group Symbols	Typical Names
Coarse Grained Soils[*]	Gravels[‡]	Clean Gravels (Little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines.
		Gravels with fines (appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixture.
			GC	Clayey gravels, gravel-sand-clay mixture.
	Sands[§]	Clean sands (Little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines.
			SP	Poorly graded sands or gravelly sands, little or no fines.
		Sands with fines (appreciable amount of fines)	SM	Silty sands, sand-silt mixture.
			SC	Clayey sands, sand-clay mixture.
Fine Grained Soils[†]		Silts and Clays Liquid limit is < 50 percent	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
		Silts and Clays Liquid limit is > 50 percent	MH	Inorganic silts, micaceous or diatomaceous fine sand or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
Highly Organic Soils			Pt	Peat and other highly organic soils.

*[redacted] More than half of material larger than No. 200 sieve size (No. 200 just visible to naked eye).
†[redacted] More than half of material smaller than No. 200 sieve size (No. 200 just visible to naked eye).
‡[redacted] More than half of coarse fraction larger than No. 4 sieve size (No. 4 ≈ 1/4 in.).
§[redacted] More than half of coarse fraction smaller than No. 4 sieve size (No. 4 ≈ 1/4 in.).

[redacted]

[redacted]

drag loading. These additional effects tend to distort and remove the cover. Compensations or reduction of these additional effects can be accomplished by using a flat rather than a sloped cover.

[redacted] The best overall protective structures require massive construction. Material such as concrete with strong, heavy blast doors represents the usual choice. Logistic requirements probably will not permit this type of structure to be constructed in the combat zone. The method most likely to be used to improve the effectiveness of field fortifications in the combat zone is one that will increase the time to peak overpressure of the blast wave. This can be achieved by providing very small doorways and an antechamber shelter. Almost any type of construction may be used with this method. Normally, if the earth cover is very thick, the roof supporting members will absorb the energy of the blast wave and the structure will not fail. A small doorway and an antechamber will modify the blast wave by "metering" through the small openings. The overpressure inside such a shelter can be reduced to about 50 percent of the incident value for rapidly decaying blast waves. However, if there is little decay during the time required to fill the chamber, the peak overpressure will not be reduced. The time to peak overpressure is approximately equal to the ratio (in feet) of chamber volume to entrance area.

11-31 Thermal Radiation Damage [redacted]

[redacted] Thermal radiation has limited effectiveness in producing damage to field fortifications. The primary effects are superficial scorching of exposed wooden revetments or reinforcements. Other exposed materials, such as sandbags, are susceptible to incident and scattered thermal radiation. Sandbags have failed at 10 cal/cm^2 from a 40 kt weapon. Light revetting materials such as burlap or pasteboard are also subject to failure if they are exposed to high levels of incident or scattered thermal radiation.

[redacted]

Problem 11-10 Calculation of Damage to Field Fortifications

[redacted] Figures 11-56 and 11-57 show the ground distance, as a function of height of burst, at which there is a 50 percent probability of severe, moderate, or light damage to various field fortifications in Nevada type soil as a result of a 1 kiloton explosion. Since there is little difference between a curve for 10 percent probability of severe damage and one for 90 percent probability of moderate damage, both may be indicated by drawing a single curve midway between the curves for 50 percent probability of moderate and severe damage. For 90 percent probability of severe damage, use one-half the yield at the same height of burst.

[redacted] Figure 11-58 shows damage levels to unrevetted foxholes and trenches in various soils.

[redacted] *Scaling:*

Figures 11-56 and 11-57: For yields other than 1 kt, scale as follows:

$$\frac{d}{d_1} = \frac{h}{h_1} = W^{1/3}$$

where d_1 and h_1 are the distance from ground zero and the height of burst respectively, for 1 kt; and d and h are the corresponding distance and height of burst for a yield of W kt.

Figure 11-58: For yields that are shown, the overpressure corresponding to a desired level of damage may be determined. The corresponding radius of damage may then be scaled from the distance and height-of-burst for 1 kt by the scaling rules indicated for Figure 2-17 through 2-22. For other yields, the impulse may be determined from Figure 11-58, and the distance and height-of-burst may be scaled by the rules indicated for Figure 2-35 and 2-36.

[redacted] *Example 1* [redacted]

Given: A 20 kt burst at 500 foot HOB.

Find: The horizontal distance at which there is a 50 percent probability of severe damage to a command post in Nevada type soil (consolidated dry sand and gravel).

[redacted]

Related Material: See paragraph 11-29.

[redacted] *Example 2* [redacted]

Given: A 100 kt burst at a height of burst of 3,710 feet.

Find: The damage to foxholes in clayey silt at a distance of 1,390 feet. (Assume near-ideal surface conditions.)

[redacted]

DNA

[redacted]

Related Material: See paragraph 11-29. See also Figures 2-17 through 2-22, and 2-35 through 2-36.

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Figure 11-56. [redacted] Machine-gun Emplacements ~~Command Posts and Personnel Shelters~~ (reference Table 11-13)—Damage by 1 kt as Function of Height of Burst and Ground Distance [redacted]

[redacted]

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Figure 11-57. [redacted] Command Posts and Personnel Shelters ~~Machine-gun Emplacements~~—Damage by 1 kt as a Function of Height of Burst and Ground Distance (reference Table 11-13) [redacted]

[redacted]

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Figure 11-58. [redacted] Damage Levels to Foxholes and Trenches in Selected Soils Versus Overpressure Impulse [redacted]

[redacted]

SECTION VII

[redacted] FIRE IN URBAN AREAS [redacted]

[redacted] INTRODUCTION [redacted]

[redacted] One of the problems common to most target areas is the danger of damage by fire. A mass fire in an urban area is one of the most devastating effects that thermal radiation can produce; moreover, the problems that must be considered in the analysis of fires in urban areas are basic to many of the aspects of thermal damage to other targets. This section considers the problems of fire in urban areas. The response of materials to thermal radiation and the combined response of materials to thermal radiation and air blast are described in Sections III and IV, Chapter 9. Ignition levels of many combustibles are also given in Section III, Chapter 9, and a discussion of survival in fire areas is also provided there. Skin burns and the effects of thermal radiation on the eyes are treated in Section II, Chapter 10, and thermal damage of special interest with respect to specific targets is treated in the various other chapters of Part 2 of this manual.

[redacted] EVOLUTION OF MASS FIRES [redacted]

[redacted] Near ground zero, a nuclear weapon burst over an area containing sufficient combustible material will inflict essentially total destruction by the combined effects of blast damage and intense fires. An analysis of damage mechanisms in this central area is unprofitable, unless the area contains targets that are hardened to nuclear bursts.

[redacted] As the distance from ground zero increases, the situation changes continuously. The extent of thermal and blast damage depends on many details, such as how buildings are constructed and whether the leaves are on the trees. The size of the area damaged by the burst depends on what happens in this critical transition region.

[redacted] Accordingly, this section is concerned principally with the response of materials to radiant exposures not exceeding a few tens of calories per square centimeter. General statements, e.g., the statement that the thermal pulse does not ignite thick, sound boards, are not usually applicable in the region of very high radiant exposure.

11-32 Ignition Points [redacted]

[redacted] Most fires that are started by thermal radiation originate in thin kindling fuels. These fires generally are not significant. Significant fires only result when fires in thin fuels spread to contiguous thicker fuels, or when thicker fuels, e.g., dry rotted wood, are ignited directly. Thus, the growth from many small, isolated ignitions to a fully developed mass fire depends on the presence of thin kindling fuels and on the presence and spacing of all combustible fuels in the area.

[redacted] In most inhabited areas, fires that are ignited indoors are more likely to develop into significant fires than fires that are ignited outdoors. Large amounts of kindling fuels are required to maintain a fire sufficiently long to ignite a sound wooden structure, and the required fuel arrangements are much more common indoors than outdoors. Ignition and burning of outdoor fuels depends heavily upon the weather, but interior fuels are relatively insensitive to weather.

[redacted] Outdoor ignitions may be important at certain times of the year in urban areas where the land is left in its wild state. Fires that start in dead weeds or in tall, dry grass on a windy day may develop into brush fires sufficiently intense to ignite houses. Areas that are poorly maintained also are likely to be susceptible to fires that start outdoors. The fuel contained in a pile of trash often is sufficient to ignite a structure with loose, weathered siding. If the air blast cracks and splinters wood siding close to the ground.

[redacted]

[redacted] the probability that outdoor fires will involve structures is increased.

11-33 Room Flashover [redacted]

[redacted] The ignition of a single item of kindling fuel in a room does not assure a significant fire. Usually, one or two combustible furnishings, such as an overstuffed chair or couch, a bed, or a wooden table, must be ignited (either directly or by adjacent kindling fuels) and must burn vigorously to cause fire spread through the room. Newspaper and other similar materials that can be ignited easily are very common interior fuels, and they are frequently found on and near furniture. In the absence of such fuels, material-covered furnishing can be ignited with a somewhat larger radiant exposure.

[redacted] Tests indicate that when fires become large enough to spread they generally will burn between 10 and 20 minutes before room flashover. Flashover is the transition that occurs when flames from a localized fire suddenly spread to fill the room. It is preceded by heating of all exposed surfaces in the room, and it occurs after combustible surfaces become sufficiently hot to emit gaseous decomposition products. The hot, combustible vapor usually accumulates near the ceiling. Ignition of this vapor increases the rate at which the fire generates heat, and the fire spreads rapidly.

11-34 Active Burning of a Structure [redacted]

[redacted] After room flashover the fire becomes intense enough to penetrate interior partitions and to spread to other rooms. The blaze from a single fire in a typical residence may be expected to reach peak intensity in roughly one hour. A larger building with 10 percent of its rooms ignited initially also will be burning most actively in about one hour. Fires caused by simultaneous ignitions in about half of the rooms of a structure ordinarily would develop more rapidly and would reach peak intensity in about 30 minutes. These times depend on building construction and fuel arrangement and are highly variable.

11-35 Fire Spread Between Structures [redacted]

[redacted] Mechanisms for fire spread from one structure or another may be conveniently classified as short-range and long-range effects.

Short-range fire spread: The direct spread of fire from one structure to another usually is caused by radiant heat transfer. The radiating portions of burning buildings emit about 4 cal cm^{-2} sec^{-1}, a value corresponding to a radiating temperature of a little more than 1,400°K. Wooden surfaces exposed to about 0.4 cal cm^{-2} sec^{-1} for extended periods of time will scorch, smoke, and eventually burst into flame. This manner of ignition is probable if the dimensions of the burning structure are as large as (or larger than) the distance to the unburned structure. Fire can also spread over short distances by convection. Hot gases from a burning structure may contact a structure that is not burning. This mechanism usually is not a determining factor in how a fire will spread in an urban area. In the absence of strong winds, radiation usually is more effective than convection. If strong winds are blowing, long-range fire spread mechanisms usually have the greatest effect on the advance of the fire front.

Long-range fire spread: Strong updrafts are produced by intense fires, and these updrafts frequently are able to lift fairly large pieces of burning wood or other fuel high into the air. If a moderate or high wind is blowing, these burning pieces may drop several hundred feet from their source. Some of them will ignite structures on which they land and may start new fires. The burning pieces are called *firebrands*, and this process of starting new fires is called *spotting*.

[redacted]

[redacted] Initially, most of the fires that are started by a nuclear burst will burn as individual fires. The airflow pattern around each fire (the convective rise of hot gases and the inflow of fresh air) is not appreciably affected by the presence of other fires nearby. However, if the area is heavily built up (so that the density of fuel is high) and a large fraction of the structures are initially ignited, the fires will tend to merge and form a single convective column that may rise to 25,000 feet or higher. When the convective column condition is reached, the fire is said to be a *mass fire*.

[redacted] A fire that covers a large area is not necessarily a mass fire. In many residential areas, the buildings are far enough apart that the available fuel will be inadequate to support mass fires. Although the conditions that will result in a mass fire are not now well-known, a high rate of fuel consumption per unit area is known to be a strongly contributing factor.

[redacted] Mass fires are of two types: *firestorms* and *conflagrations*. The form that a mass fire assumes will depend on whether the airflow in the region is dominated by convection forces or by ambient wind forces.

[redacted] A firestorm is a mass fire with an essentially stationary fire front. It is the more likely type of mass fire when wind is light. The overall airflow pattern tends to be roughly symmetrical. Strong winds blow into the fire at its edge to replace the air in the rising convection column. In some cases, fire whirlwinds up to 1/2 mile in diameter may develop, producing winds of 100 mph or more.

[redacted] Once a firestorm develops, any unburned object within the perimeter is soon surrounded by flame and is ignited. Destruction within the firestorm area is, for this reason, essentially complete. On the other hand, by blowing against the fire front, the same strong winds that keep the fire burning at high intensity help to protect the area that was not ignited initially. Both flames and firebrands are carried inward, and the winds help to cool buildings outside the fire. Thus, the danger of outward spread is reduced.

[redacted] The term "conflagration" often is used loosely to mean any large fire, particularly one that spreads to a number of buildings. In this manual the term is used in a stricter sense to mean a mass fire driven by the wind. Being a mass fire, it involves a large area that is burning intensely.

[redacted] A conflagration results when the ambient wind is strong enough to override the winds caused by convection. It is characterized by an advancing fire front, particularly on the downwind side of the fire. The front advances by direct contact of the flame with unburned structures and by the spotting of numerous fires by firebrands. The wind blowing outward from the fire in the downwind direction is often much stronger than the ambient wind.

[redacted] Many conflagrations have occurred in peacetime. Examples include intense forest fires and large urban fires, such as the great Chicago fire of 1871. Conflagrations often burn until they are stopped by a natural firebreak or until the winds die down or change direction.

[redacted] When winds are strong, conflagrations are likely to advance in surges, traveling at rates up to 3 miles per hour. The same fires would advance at about 1/10 to 1/2 mile per hour without the support of strong winds, depending on fuel, weather, and topography.

[redacted] The spectacular increase in intensity of a fire that reaches the proportions of a mass fire suggests that further increases in size might produce fires that are even hotter, and, in the case of conflagrations, would spread even more rapidly; however, this is not the case. In a firestorm, the strong inward winds that maintain an intense level of combustion at the fire's edge penetrate only about a half mile into the fire area. Deeper inside the firestorm, air that is required to feed

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[redacted]

[redacted]

the fire comes from above. Conditions in this central region are no different than they would be if the fire were infinitely large.

[redacted] After a large nuclear burst, several mass fires might be scattered throughout a large burning area. Initially, some areas may not be burning. These would be areas that lie in the shadow of a hill or cloud, or areas where large deciduous trees screen nearly all structures. Even if the fire spreads widely, areas such as these may not be fully ignited before other areas are burned out. Large areas with little fuel, e.g., parks, probably will be safe from fire during the entire period. Active burning time in most urban areas would be between 1/2 and 2 hours at any given location.

[redacted] ESTIMATION AND CONTROL OF THERMAL DAMAGE [redacted]

[redacted] Since the ignition and spread of fires depend on the interactions of a large number of variables, it usually is not possible to make accurate predictions of fire damage. Even a detailed study of a particular urban area cannot predict such vital factors as shifting of winds (which may be affected by the fire itself). The ignition pattern produced by a nuclear burst can be predicted with some degree of confidence, however, and the evolution of the fire can be described in general terms.

11-37 Radiant Exposure Thresholds [redacted]

[redacted] The maximum possible radius for direct thermal damage by fire is the radius at which light combustibles can be ignited. The thermal response of various materials is discussed in Section III, Chapter 9; however, for most purposes a detailed examination of the properties of many materials is not necessary. Much of the following discussion is based on the properties of a single kindling fuel, newspaper. This is a light combustible that is commonly found in urban areas; moreover, its properties are similar to those of many other materials that are easily ignited. Its ignition thresholds have been investigated extensively.

[redacted] The radiant exposure required to ignite newspaper depends on several factors. Pictures are usually darker than text printing; therefore, picture areas will absorb more heat than will areas covered by text. Variations in moisture content affect the radiant exposure required for ignition (see Section III, Chapter 9).

[redacted] A single sheet of newspaper is sometimes more difficult to ignite than the outer sheet of a loosely folded newspaper. Heat loss from a single sheet takes place at both front and back surfaces. If, on the other hand, a newspaper is loosely folded, heat from the back side of the outer sheet is largely trapped between the first and second sheets.

[redacted] Data suggest that this last effect is unimportant for yields below about 10 Mt. Pulses of shorter duration than the pulse from a 10 Mt burst allow insufficient time for heat loss from the back side of exposed newspaper samples; consequently, the insulating layers have only a small effect on the temperature reached during short pulses.

[redacted] Geometrically complex shapes, e.g., crumpled newspaper, ignite about as easily as folded newspaper.

[redacted] Experiments indicate that the ignition thresholds for folded or crumpled newspaper for very long pulses (corresponding to 1,000 Mt or more) approaches about 2/3 of the thresholds for short pulses; however, specific quantitative relations have not been developed. Figure 11-59 shows the range of radiant exposures required to ignite newspaper.

[redacted] Although combustibles indoors are likely to be drier than those outdoors (particularly in cold weather), a higher radiant exposure often is required to start fires indoors. Windows that allow exposure of indoor combustibles can produce attenuation in three different ways. First, a

[redacted]

[redacted]

window admits only the direct flux plus that portion of the scattered flux that comes from the portions of the sky that are visible through the window from the position of the target. Second, window screens and panes of glass attenuate thermal energy. Third, the relative positions of the fireball, the window, the target, and nearby trees or structures often provide shielding that allows the target to be exposed to only a portion of the fireball radiation.

[redacted] Data to estimate the protection that windows provide from scattered flux are sketchy. Computer codes can be used to determine direction and spectral distribution as well as the amount of radiation reaching a given point; however, complete results checked with experimental data are not available.

[redacted] Attenuation by glass and screen can be estimated easily. A pane of glass generally reduces thermal energy by about 20 percent. Very little of this loss is in the glass itself; most of it is caused by reflection of energy at the surfaces. Therefore, attenuation is nearly independent of the thickness of the glass. The attenuation depends on the angle of incidence. A transmission factor of 80 percent is a good approximation for angles between 0° and 60°, but at grazing angles the amount of energy reflected is greater and transmission drops rapidly. Window screens block about one-third of the incident radiation.

Problem 11-11 Calculation of the Radiant Exposure to Ignite Combustibles Indoors

[redacted] Figure 11-59 shows the range of radiant exposures required to ignite newspaper. Over most of the graph, the bottom of the band estimates the values for dark-printed sheets and the top of the band estimates thresholds for sheets with ordinary text. At the highest yields shown, the bottom of the band represents dark-printed single sheets or folded newspaper with ordinary text printing, whichever is lower; the top of the band represents single sheets with ordinary text. Moisture content is that corresponding to a relative humidity between 40 percent and 50 percent. When relative humidity is close to zero, the ignition thresholds are about 80 percent of the values shown; when it is close to 100 percent the thresholds are about 120 percent of the values shown.

[redacted] *Example* [redacted]

Given: A 1 Mt weapon burst over an urban area during cold weather.

Find. The minimum radiant exposure required to ignite light combustibles indoors.

Solution: Figure 11-59 shows that a 1 Mt burst must produce a radiant exposure of about 6.5 cal/cm^2 to ignite folded newspaper in air of about 50 percent relative humidity. During cold weather, humidity in heated interiors is usually quite low; therefore ignitions can be expected with about 80 percent of 6.5 cal/cm^2, or 5.2 cal/cm^2. It is reasonable to expect the protection of at least a single pane of glass with an attenuation of 20 percent in virtually all exposures.

Answer: Considering both the low relative humidity indoors and the minimal expected attenuation of a single pane of glass, the free-field radiant exposure must be about 6.5 cal/cm^2 to ignite kindling fuels indoors. At the range where this free-field radiant exposure occurs, indoor fires will be started only where conditions are such that materials that are ignited easily receive full exposure to the incoming radiation. If weather conditions are such that a substantial fraction of the 6.5 cal/cm^2 arrives as scattered radiation, few ignitions are to be expected. Therefore, the radius at which radiant exposure is 6.5 cal/cm^2 is probably an overestimate of the radius at which the danger of fires is severe.

Related Materials: See paragraph 11-37.

[redacted]

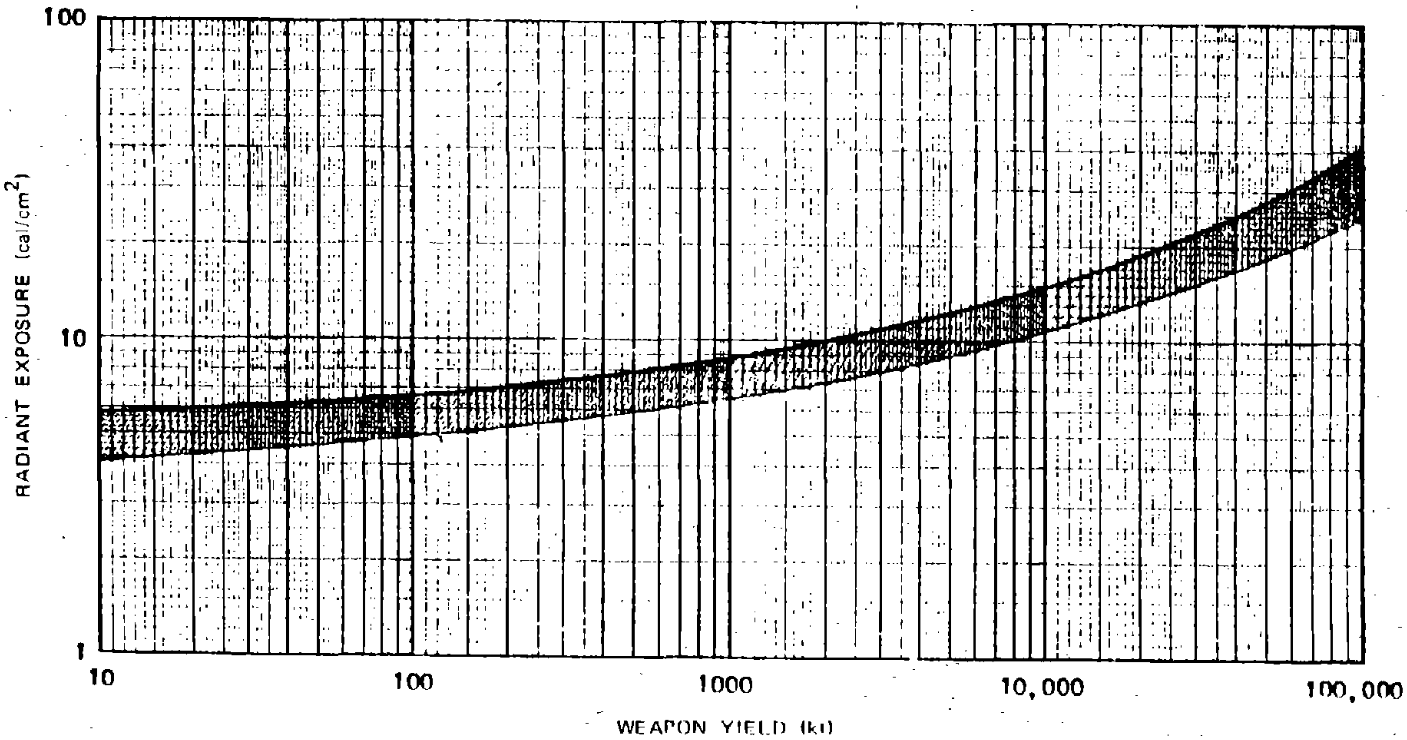

Figure 11-59. [redacted] Ignition Thresholds for Newspaper [redacted]

[redacted]

11-38 Fire Radii [redacted]

[redacted] Methods for setting limits on the radius within which fire may be expected to be a serious problem can be illustrated best by analysis of specific situations. Two examples chosen from the Five-City Study conducted by the Office of Civil Defense are described in succeeding paragraphs.

11-39 The San Jose Study [redacted]

[redacted] Input data for the study of San Jose, California, were obtained by trained firemen, who examined a large number of residences and other structures in the San Jose area. Potential ignition sites were identified. The arrangement of other fuels provided the basis for a judgment as to whether an ignition would lead to a significant fire. The locations of both ignition sites and windows were noted. Both written and photographic records were made.

[redacted] A statistical analysis of the effects of a hypothetical nuclear burst was then performed. The assumed conditions were:

Yield	5 Mt
Burst height	14,500 feet
Time of year	August
Visual range	22 miles
Cloud cover	1/10 scattered clouds, ceiling unlimited
Wind	Less than 10 mph from the west
Burst location	About 13 miles north of San Jose business district

[redacted] During August, the relative humidity may be expected to be moderately low in San Jose. The radiant exposure required to light folded newspaper is about 9 cal/cm^2 when humidity is 40-50 percent and nearer 8 cal/cm^2 for the conditions in San Jose in August. This figure might apply to a fully exposed target behind an unscreened, open window.

[redacted] The effects that were calculated for the 5 Mt weapon are shown in Figure 11-60. Sixteen miles from ground zero, where the radiant exposure is 10 cal/cm^2, significant fires will be started in about 2 percent of single-family residences.* At 13 miles, radiant exposure rises to 17 cal/cm^2, and over 10 percent of the residences would sustain fires that, if uncontrolled, would destroy the structure. At shorter ranges, the fraction of homes ignited increases rapidly, and major fire losses are almost assured.

[redacted] The long-range extent of fire damage may be estimated from the top scale in Figure 11-60 for cases where the firespread potential is moderate. If the fire spreads principally only within blocks, the number of blocks in which at least one structure is ignited is a good indicator of the number of blocks that will eventually be burned out. The upper scale in Figure 11-60 does not represent conditions in San Jose exactly; it represents a hypothetical situation in which each block contains 20 structures. The number of blocks containing a significant fire drops much more abruptly with increasing range than the fraction of structures containing a significant fire. This indicates that, in the absence of fire-control measures and with high firespread probability between adjacent houses, the radius of the completely burned-out area is fairly well defined at about 13 miles.

[redacted] An analysis of blast damage is required to present a complete picture. Fifteen miles from ground zero, the overpressure of 1.7 psi has a 50 percent probability of producing light damage: windows and doors may be blown in; wall framing, rafters, and interior partitions may be cracked. At 10-1/2 miles, moderate blast damage is expected: wall framing may be badly cracked, the roof may be damaged, and interior partitions may be blown down. Structures in

*[redacted] The radiant exposure data in Figure 11-60 cannot be confirmed by the methods set forth in Chapter 3. Thermal partition and transmittance data used in the Five-City Study are not identical to the data presented in Chapter 3.

[redacted]

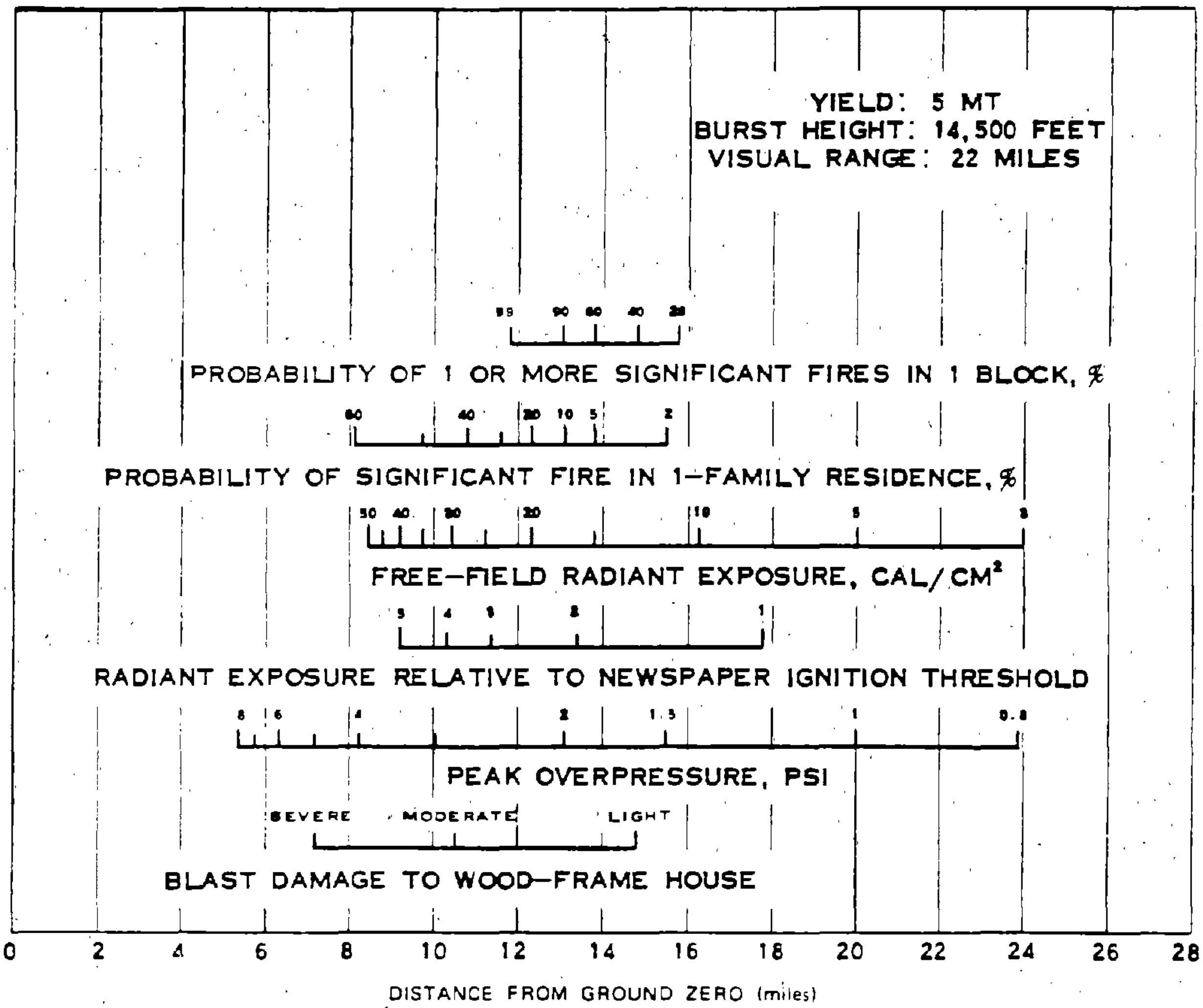

Figure 11-60. [redacted] Graphical Summary of San Jose Study [redacted]

[redacted]

[redacted]

this latter condition will not be resistant to fires that start indoors or outdoors. Additionally, splintered pieces of wood are likely to fall where they can contribute to outdoor fires. At 7 miles, the overpressure has a 50 percent probability of producing severe damage to homes: the frames are shattered and most structures will collapse. Fires are expected to develop and to spread rapidly. Blast damage will impede efforts to extinguish fires while they are still small.

[redacted] The San Jose study reveals a situation in which the uncertainty as to the amount of fire damage is relatively small. Between 12 and 13 miles from ground zero, the number of fires is sufficiently high that a few of the fires may be expected to get out of control. At the same distance, blast damage is severe enough to contribute substantially to the rate of firespread.

[redacted] The San Jose analysis used an approximation that is conservative from the point of view of the defense. The calculation of the value of direct radiant flux was based simply on transmittance. Since transmittance indicates the amount of direct plus scattered radiation, the amount of thermal energy passing through a window and striking a specific object would be, in most instances, less than the amount that was calculated. Because of the strong forward-scattering properties of haze particles, this approximation involves a smaller error than would exist if the scattered radiation had a uniform intensity from all parts of the sky.

[redacted] Structures other than residences also were studied. Significant fires were predicted in half of the structures in the central business district, even though the ground range is about 13 miles, the range at which only 10 percent of the single-family houses sustain significant fires. The difference lies chiefly in the sizes of the structures. Buildings in the business district are relatively tall so more of them have an unobstructed view of the fireball. The average building in the business district has more windows than the average residence, so there are more potential ignition sites per building.

[redacted] Under the conditions assumed in the study, a firestorm would be probable in the central business district of San Jose, and untenable conditions would be expected within 1/2 to 1 hour of the detonation. As a result of the light wind, fires in residential areas would be expected to be governed by short-range spread mechanisms. Although entire blocks may burn out, fires in residential areas are not expected to be so intense that survival within the fire area would be a severe problem. Probabilities that were calculated for across-the-street spread from one block to another and back-to-back spread within blocks are low for most of the areas; therefore, the fire is not expected to spread much beyond the area ignited.

11-40 The New Orleans Study [redacted]

[redacted] Another portion of the Five-City Study illustrates a situation in which thermal damage is relatively difficult to calculate. The assumed conditions were:

Yield	10 Mt
Burst height	Surface burst
Time of year	August
Visual range	15 miles
Cloud cover	None
Wind	Less than 10 mph from the south
Burst location	Central business district

[redacted] The effects predicted from such a burst are shown in Figure 11-61. The number of fires started indoors by thermal radiation is predicted to be low beyond the range for 6-psi overpressure, even though the free-field radiant exposure at this distance (5.3 miles from ground zero) has the very high value of 160 cal/cm^2. The low incidence of predicted indoor ignitions results from the low elevation angle of the fireball. The artificial horizon of trees and buildings obscures the fireball from most residential windows (the

[redacted]

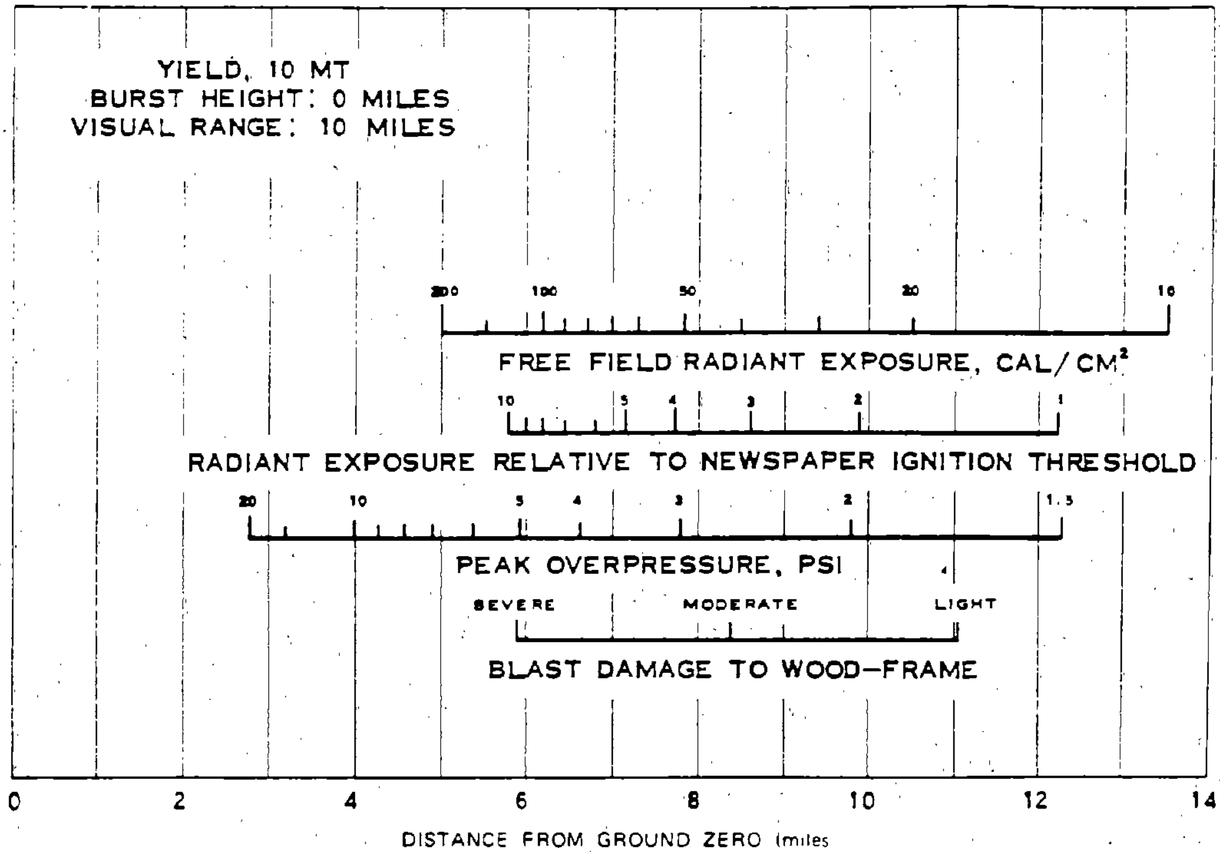

Figure 11-61. [redacted] Graphical Summary of New Orleans Study [redacted]

[redacted]

[redacted]

average elevation angle of the artificial horizon is about 6° for New Orleans).

[redacted] In the San Jose Study, the assumed burst height of 14,500 feet places the fireball where it can shine through many windows and can ignite a large number of fires indoors. Mechanisms for starting fires outdoors also exist, but the ignition patterns are largely determined by the many indoor fires.

[redacted] In the New Orleans Study, very few indoor fires are predicted, and outdoor fires assume a much higher relative importance. Unfortunately, the spread of most outdoor fires depends on mechanisms for which reliable prediction techniques are not established.

[redacted] Many secondary fires would be started by blast damage alone. Other significant fires would result from a combination of blast and thermal damage, e.g., from splintered wood being thrown on small fires started in thin fuels. Flammable roofing materials might provide ignition sites. Scattered thermal energy, not separately accounted for in the transmittance calculations used in the Five-City Study, will add to the small amount of direct energy that enters windows of houses and will help to ignite indoor materials where no fires are predicted.

[redacted] Much of the fire danger depends on blast-thermal interactions. Fire damage is expected to be light where blast damage is light. Where blast damage is heavy, scattered fuel will aid the spread of fire, and even a few primary fires probably would destroy the area. The principal uncertainties lie in the region of moderate blast damage.

11-41 Effects of Clouds [redacted]

[redacted] Heating of air by the blast wave clears the air of small water droplets for some distance beyond the edge of the fireball. A burst at the bottom edge of a cloud layer clears a region through which thermal energy can propagate away from the fireball. The clouds above the burst form a reflecting surface that carries thermal energy outward from the burst. If the radiation arrives primarily as scattered flux, the situation differs from the receipt of direct flux in several respects:

- Since the incident flux no longer arrives as a direct beam, its ability to penetrate deeply into a room at essentially full strength is lost.
- That portion of the scattered flux that arrives at a window at nearly grazing incidence is strongly reflected by the glass. Roof overhangs and the window frame provide further help to eliminate this portion of the scattered flux. For equal radiant exposures*, the amount of scattered energy that can penetrate a room through a window is roughly 2/3 of the energy that could penetrate the room if all the energy were received as a direct beam through a window facing the burst.
- Radiant exposure at a window is less critically dependent on location of trees, other structures, etc.
- Thermal energy enters all windows, not just those facing the burst.
- Interiors of the rooms are more uniformly illuminated, because the energy spreads out as it propagates past the window opening.

[redacted] The thermal energy reaches many more indoor ignition sites when it arrives as scattered flux, but at all of these sites the radiant exposure is considerably less than the free-field radiant exposure.

[redacted] The primary targets for scattered thermal radiation are curtains, drapes, and window shades. After these targets ignite they may be

*[redacted] Free-field radiant exposures are considered equal when flat surfaces optimally oriented (usually this means that the surfaces are almost directly facing the burst point) receive equal radiant exposures.

[redacted]

[redacted]

blown directly across the room by the blast wave. Significant fires may result if furniture placed opposite windows provides an adequate, properly located fuel supply. Because of the way in which radiant exposure decreases with distance from the window, potential targets deeper in the room, e.g., a newspaper lying on a chair, become much less likely to ignite.

[redacted] The Five-City Study suggests that closed curtains and shades exposed to direct flux are about half as effective in starting fires as other objects within the room when the curtains and shades are open. The figures are controversial, because it is difficult to predict the results of throwing burning curtains and shades across a room. However, with scattered flux entering windows on all four sides of a house, curtains and shades provide about as many sites per house for significant fires as other targets exposed to direct flux.

[redacted] On the basis of all of these considerations, indoor ignitions from strongly scattered radiation are expected to occur in less than 10 percent of the exposed structures, unless free-field radiant exposure exceeds two or three times the ignition threshold for newspaper. Above this level, the number of significant fires is expected to increase rapidly.

[redacted]

[redacted]

[redacted] The information provided in the preceding paragraphs of this section provides a means for making at least crude estimates of the extent of fire damage in urban areas.

[redacted] *Example* [redacted]

Given: [redacted] DNA (b)(3)

Find: Thermal damage levels within the city.

Solution: A number of unknown factors make this example a difficult one; however, this one is typical of many practical problems. This example illustrates methods of applying the results of the Five-City Study; it also introduces additional factors that this particular problem requires. Separate discussions are provided for damage probabilities on a clear day, and damage probabilities on a cloudy day.

Damage Probabilities on a Clear Day

Figure 11-59 shows that the lowest ignition threshold for newspaper is [redacted] The San Jose Study suggests DNA (b)(3) that thermal effects start to become important at about twice this threshold value [redacted] The relations between this threshold and other parameters for the assumed yield and burst heights are shown in Figure 11-62. Radiant exposures have been calculated for a clear day.

[redacted] creates thermal effects that compare closely with those predicted in the San Jose Study. As a first approximation, therefore, similar damage levels would be assumed. For example, where the free-field radiant exposure is twice the ignition threshold of newspaper, about 15 percent of the one-family residences would sustain significant fires. At roughly the same range, 30 percent of larger structures such as apartment houses and office buildings would sustain significant fires.

A better approximation should consider differences between Russian and American cities. [redacted] These factors cannot be evaluated accurately; therefore, they are considered as uncertainties in the analysis rather than as the basis for correction factors.

[redacted] presents almost as difficult a prediction problem as the New Orleans Study. The fireball [redacted] DNA (b)(3) s so low that the amount of thermal energy that would enter windows would be modified by the artificial horizon of other structures and trees. This artificial horizon is probably lower [redacted] particularly during the winter months. Nevertheless, a few fires would be expected in the one-family residences that are far enough from the burst to avoid substantial blast damage. Taller buildings would probably sustain nearly as much damage as the San Jose Study suggests.

Damage Probabilities on a Cloudy Day

Typical cloud cover [redacted] during the winter months is of the stratiform, inver-

[redacted]

[redacted]

[redacted]

sion type. These are low, continuous clouds (the bottom is often as low as 400 feet), and visual range below the cloud layer is likely to be on the order of 10,000 feet. These clouds offer protection from thermal energy if the burst is a few thousand feet above the surface, but they may enhance thermal damage from bursts near the surface, as described in paragraph 11-41. The clouds above the burst and the probable snow layer on the ground form reflecting surfaces of a duct that carries thermal energy outward from the burst.

Confining the radiation in this manner enhances the thermal radiation transmitted to targets on the ground. As indicated in Chapter 3, this enhancement could be as much as 2.25 times the radiant exposure that would be received in the absence of these reflecting layers. At the same time, the hazy atmosphere between the snow and the cloud layers produces some attenuation. The principal effect of this type of atmosphere is thus not so much a change in free-field radiant exposure as a change in the direction of the incident radiation. It no longer arrives principally as a direct beam; it is scattered toward the target from many directions. Thus, on the basis of the discussion in paragraph 11-41, interior ignitions would be expected to occur in less than 10 percent of exposed structures unless the free-field radiant exposure exceeds two or three times the ignition threshold for newspaper.

Effects of Multiple Bursts

Blast overpressures exceeding 1/2 psi will break most of the windows facing the burst. Overpressures exceeding 1-1/2 psi will break nearly all windows. Houses thus damaged are apt to be more susceptible to thermal radiation from a second burst, because the first burst has removed a source of attenuation. Since a window pane passes only about 80 percent of normally incident thermal energy, the radiant exposures required to produce a given damage level can be reduced by 20 percent when window panes are removed.

Concurrent effects include the removal of curtains, one of the targets of thermal radiation, but also one of the sources of attenuation for other ignition sites within the room. The net effect of these changes is believed to be a further increase in thermal vulnerability on clear days and a decrease in thermal vulnerability on hazy days.

Splintered wood produced by the blast wave may add numerous outdoor ignition sites at points where ignitions can result in significant fires. This effect is expected to be particularly important in areas where wooden shingles are common.

Kindling fuels that are too moist to be ignited by the first thermal pulse may be dry enough to be ignited by a second pulse. However, if the fuel is initially wet rather than just moist, several exposures may be required.

Scorching by the first pulse also increases target vulnerability. Since the surface of the material is blackened, it will absorb a greater fraction of the incident thermal energy from a second pulse. Under favorable conditions, this effect would be expected to reduce ignition thresholds by about one-third.

[redacted]

[redacted]

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Figure 11-62. [redacted] Graphical Analysis of the Thermal and Blast Effects of Two Nuclear Bursts [redacted]

[redacted]

[redacted]

11-42 Firestorm Criteria [redacted]

[redacted] The examples reviewed in preceding paragraphs provide indications of areas that are likely to be destroyed by fire, but the likelihood of a mass fire has only been discussed briefly. The distinction is important, because it affects the probability of survival for people in the area. A large number of individual fires still leave relatively cool areas where fresh air is available, but a mass fire increases the danger of poisonous gases for those who remain in inadequate shelters and the danger of excessive heat for those who try to escape the area.

[redacted] The criteria for mass fires are still the subject of controversy; nevertheless, tentative criteria have been proposed. These criteria provide useful information if it is realized that they are only rough guides.

[redacted] The intensity of a large fire depends, in part, on the average amount of combustible material per unit area. In Hamburg, where 45 percent of the firestorm area was covered by buildings containing about 70 lbs/ft^2 of fuel, the average fuel loading was 32 lbs/ft^2. A strong firestorm was produced in the area from the World War II incendiary bomb raid.

[redacted] In Hiroshima, the average fuel loading is estimated to have been 8 lbs/ft^2. The fire resulting from the nuclear burst clearly showed firestorm characteristics; however, there is some question whether the fire was of sufficient intensity to warrant classifying it as a true firestorm. Energy release rates that have been calculated place Hiroshima in a class with Japanese and German cities that did not develop firestorms.

[redacted] If the Hiroshima fire is classed as a firestorm, the fuel-loading criterion may be set at 8 lbs/ft^2; if it is not so classified, the criterion is uncertain. No firestorm data are available for fuel loadings between about 8 lbs/ft^2 and 30 lbs/ft^2. A suitable figure chosen to fall between these limits can be established only by relatively uncertain theoretical considerations.

[redacted] In areas where wood is used extensively in construction, the amount of fuel may be estimated to be about 15 lbs/ft^2 for each story of structure. Estimates based on this rule of thumb are subject to large errors in individual cases, but reasonable accuracy usually is obtained in estimates of average fuel loading over large areas that include both residential and commercial structures.

[redacted] At some point in the evolution of a large fire, a large fraction of the structures must be burning simultaneously if the fire is to become a firestorm. If this point is reached at all, it is generally reached within a short time, typically an hour or two, after the fire starts. If the fire develops more slowly, the number of structures that are burned out limits the peak intensity of the fire, even in areas that are completely destroyed by the fire.

[redacted] A firestorm requires either ignition of a large fraction of the structures in the area or rapid firespread. However, the probability that a rapidly spreading fire will become a firestorm is low. The rapid spread of fires from a few ignited structures is likely only in a high wind, but a high wind also can prevent a firestorm from developing. Therefore, a firestorm is unlikely if the fraction of structures initially ignited is below about 50 percent.

[redacted] In some cases, the development of a firestorm depends on whether the convective forces produced by the fire are strong enough to override ambient winds. If surface winds exceed about 8 miles per hour, a firestorm is unlikely. In such cases, the fire is likely to spread as a conflagration.

[redacted] Almost without exception, the firestorms that have occurred as a result of bombing have covered areas that exceed a square mile. Although test burns covering smaller areas have shown some firestorm properties, fires in urban areas seem to require areas of half a square mile or larger before the rate of energy release is great

[redacted]

[redacted] enough to produce the strong convection column and surface wind pattern that characterize a firestorm.

[redacted] Rain and snow are not serious deterrents to mass fires in urban areas. The principal sources of fuel are indoors and therefore are dry. World War II experience indicates that incendiary raids carried out during rainstorms produced about 80 percent of the damage that would have been produced during dry weather. Reduction in damage was caused primarily by a reduction in the number of fires that started. After the fires grew enough to destroy the structures that contained them, the rain had no significant effect.

[redacted] In a nuclear attack, rain would reduce the number of outdoor ignition sites, but the number of indoor ignitions would depend principally on radiant exposure. However, the atmospheric conditions associated with rain usually would reduce the radiant energy reaching a particular target.

[redacted] The conditions favorable to the formation of firestorms are *tentatively* estimated as:

- Fuel loading of at least 8 pounds of combustibles per square foot if the Hiroshima fire is classed as a firestorm; between 8 and 30 lbs/ft^2 if it is not so classified.
- Half of the structures ignited initially.
- Surface wind 8 mph or less at the time of attack.
- Area exceeding 1/2 square mile.

The list is not complete. For example, the meteorological condition known as an unstable atmosphere might be conducive to strong convective action. In addition, the criteria must, for the present, be recognized as educated guesses, based largely on evidence reconstructed from the firestorms of World War II.

11-43 Conflagrations [redacted]

[redacted] The complexity of conflagrations prevents the identification of simple criteria for their prediction. High winds, by fanning burning structures and causing rapid spread of the fire, are known to be an important factor. The source of the high winds on the downwind side of the fire is often the fire itself. Horizontal winds blowing out of the fire that are much stronger than ambient winds may be produced.

[redacted] A major factor in the rate of spread of the fire is the ease by which unburned structures can be ignited. For example, wooden shingles have been a major contributing factor to many peacetime conflagrations. This type of roof is ignited readily by firebrands; moreover, it is capable of supplying large quantities of firebrands for further spread of the fire.

[redacted] As a result of the high rate of spread of fire on windy days, development of a conflagration would not require that a large fraction of structures be ignited initially.

[redacted] Since the intensity of a fire depends on the energy release per unit area per unit time, high fuel loading is expected to produce the same untenable conditions in a conflagration that it produces in a firestorm. Fuel loading is not high in typical urban areas. A fire in such an area would be simply a wind-driven fire and not a conflagration. Survival should be possible for a large fraction of the people trapped by such a fire.

[redacted] The destructive nature of either a conflagration or a large wind-driven fire lies in its ability to spread until the fuel supply is exhausted. The fire can cause enormous property damage, but, since the fire advances slowly into unburned areas, the threat to life is relatively minor in areas that are not ignited initially.

11-44 Fire Control [redacted]

[redacted] Under the conditions of a nuclear attack, conventional firefighting can be expected to have very limited effectiveness. The large number of fires, the limited number of trained

[redacted]

[redacted]

personnel, and the probable interruption of water supplies and communication facilities present overwhelming problems.

[redacted] A first-aid type of firefighting could, however, be extremely effective. A large fraction of the fires ignited by thermal radiation could be extinguished easily in the first ten minutes or so, i.e., before the time of flashover. Some fires can be stamped out, and a gallon or two of water is sufficient to extinguish other small fires. In areas where one structure in ten is ignited, such action could have a strong effect on the ultimate level of damage.

[redacted] Most of the firefighting would have to be done by the occupants of the structure involved. The time required to check neighboring houses for fires is comparable to the time required for a fire to get out of control.

[redacted] Extinguishing individual fires also has considerable value in areas sustaining a large number of significant fires. Even though some fires will inevitably be missed, both the rate at which the area fire builds up and the peak intensity of the area fire will be reduced if some of the initial fires are extinguished. This tactic probably will result in a small reduction of property damage, but a large reduction of the threat to survival. Extinguishing individual fires probably cannot be expected in areas of severe blast damage since there will be many immediate concerns other than the fires.

[redacted]

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